IN-SITU STRENGTH DEFORMATION CHARACTERISTICS OF ALLUVIAL SOILS

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CERTIFICATE

Certified that the thesis entitled 'In-Situ Strength-Deformation Characteristics of Alluvial Soils' is a record of work carried out by Mr. Yogesh Singh under my supervision and that it has not been submitted elsewhere for a degree.

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ABSTRACT

This investigation deals with determination of in-situ strength-deformation-permeability characteristics of alluvial soils on IITK Campus. Results of pressuremeter tests (sub-soil-deformeter) are presented and compared with other in-situ test data available for these soils.

Correlations between pressuremeter test values of limit pressure and modulus of deformation with results of standard penetration tests, static cone penetration tests and plate load tests are established. Bearing capacity values of an isolated square footing (1mx1m) as computed from pressuremeter test data at depths are shown to agree well with the values computed from static cone penetration test data.

It is shown that 2m to 6m and 0.8m to 5m water table fluctuations for two sites on campus predict values of over consolidation ratio, which agree well with those computed from oedometer test data on undisturbed samples obtained from different depths. Undrained strength profile for these lightly over consolidated deposits is given.

LIST OF SYMBOLS

A_f	:	Pore pressure parameter at failure
B	:	Diameter of test plate
C_c	:	Compression index
C_u	:	Undrained cohesion or undrained compressive strength
C'	:	Drained cohesion
e_o	:	Void ratio in-situ
E	:	Modulus of deformation
E_u	:	Undrained modulus of deformation
E'	:	Drained modulus of deformation
f_s	:	Average side friction
FR	:	Friction ratio
F	:	Factor of safety
G.L.	:	Ground level
I_p	:	Plasticity index
K_o	:	Coefficient of earth pressure at rest
k_s	:	Modulus of subgrade reaction
k	:	Coefficient of permeability
k_h	:	Coefficient of permeability in horizontal direction
k_v	:	Coefficient of permeability in vertical direction
m_v	:	Coefficient of volume compressibility
N	:	Standard penetration number (blow count/30 cm of penetration of standard split spoon)

OCR	:	Over consolidation ratio
p_1	:	Limit pressure
q_u	:	Ultimate bearing capacity
q_{all}	:	Allowable bearing capacity
q	:	Load intensity
q_c	:	Cone resistance or point resistance
SPT	:	Standard penetration test
SCPT	:	Static cone penetration test
W_n	:	Natural moisture content
W_p	:	Plastic limit
W_L	:	Liquid limit
ϕ	:	Angle of internal friction of soil
ϕ'	:	Drained angle of internal friction of soil
ν	:	Poisson's ratio
ν'	:	Drained Poisson's ratio
ρ	:	Settlement under centre of plate
γ	:	Unit weight of soil

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CHAPTER 1

INTRODUCTION

In their practice, civil engineers recognise the need for in-situ investigations prior to design and construction of any engineering structure. This need has received increasing attention due to the following reasons.

(1) It gives a realistic picture of in-situ conditions and the pattern and structure of geological strata.

(2) Speed and economy.

(3) Circumvention of the difficulties of sampling and simulation of field conditions in the laboratory while testing.

(4) Averaging of 'mass' characteristics of heterogenous materials.

From results of these in-situ tests, it is possible to provide information for design before construction begins and to obtain observations required for monitoring the behaviour of the structure during construction.

The potential value of these results is such that in-situ investigation should be more widely adopted in practice.

In-situ testing of alluvial soils has been carried out mostly through the use of SPT, SCPT and plate load tests. For campus soils, Jain (1977) and Jain (1978) have reported detailed results of such investigations. Currently, pressuremeter has been widely used to obtain both the strength and deformation in-situ characteristics of soils at depth (Menard 1963, Gibson and Anderson 1961, Hobbs and Dixon 1969). While SPT and SCPT basically give strength characteristics only , with the plate load test, we do get a definite relationship between the load and deformation but this test is confined to shallow depths ; the pressuremeter provides the much needed additional information on deformation characteristics at depth.

The present investigation was planned with the following objectives in mind :

1. To check the working and applicability of the sub-soil deformeter variety of the pressuremeter manufactured by Associate Instrument Manufacturers (India) Pvt. Ltd. for alluvial deposits.

2. To determine the strength and deformation profile with depth for alluvial soils with the help of pressuremeter tests and other data available from SPT, SCPT and plate load tests. An attempt has also been made to measure field permeability of these

deposits and compare with laboratory results on undisturbed samples. Oedometer tests on undisturbed samples have been used to estimate OCR and compare with the values predicted on the basis of water-table fluctuations in these deposits.

(3) To establish correlations between pressuremeter test data and those found from other in-situ tests.

CHAPTER 2

TEST SITE AND DETAILS OF TESTS CONDUCTED

2.1 THE SITE

IITK Campus is located on alluvial deposits of the river Ganges. A major content of this alluviam is silt but in some places there is low percentage of clay and sand also. It has been reported by Jain (1977) that large annual fluctuations of water-table take place which causes considerable amount of variations in effective stresses on soil particles. As the water table goes down, the soil gets subjected to suction pressure which results in development of negative pore-pressure and an increase in the effective stresses. This makes the soil behave as a 'lightly over-consolidated soil'. Due to these fluctuations in water table calcareous material has been brought up and deposited as concretions, locally known as 'kankar'. The distribution of kankar is quite erratic and thus laboratory testing and in-situ testing of these materials are likely to give non-representative results.

Earlier studies on in-situ and lab testing of alluvial soils by Jain (1977), Srivastava (1977) and Jain (1978) were conducted near the southern end of the

campus ,site B (Fig. 2.1). The present work was undertaken at site A (Fig. 2.1). As shown by earlier studies at site B, large fluctuations of water-table have been observed whereas in the academic area of the campus (site A), water-table is generally below 2.0 m even when the water-table at site B is at 0.8 m from ground surface. Due to scarcity of rainfall during the year of this study, water-table was not observed even upto 4.0 m depth.

2.2 DETAILS OF TESTS CONDUCTED

The following field tests were conducted:

1. Pressuremeter test
2. Standard penetration test
3. Static cone penetration test
4. Plate load test (at depth)
5. In-situ permeability test

In addition to these field tests, undisturbed tube samples were also obtained from a depth of 4.0 m and the following tests were conducted in the laboratory.

1. Oedometer test on horizontal and vertical samples.
2. Lab determination of permeability on horizontal and vertical samples.

Representative samples at different depths in all the bore holes were tested for determination of natural

moisture content, Atterberg limits and grain size analysis for the purpose of soil classification.

2.2.1 Pressuremeter Test

Six bore holes were made to conduct pressuremeter tests at site. In bore hole no. 1 and 2, tests were conducted at 1.0 m, 2.0 m, 3.0 m and 4.0 m in a sequence of first conducting the test at that depth and then extending the bore hole to the next test depth. In bore holes 3, 4, 5 and 6, tests were performed at 1.0 m, 2.0 m, 3.0m and 4.0 m respectively. This was done to find out the effect of one test on the next test, conducted in the same bore hole.

2.2.1.1 Principle

The analytical principle of cylindrical cavities expanding under radial stress has provided the basis for its development (Palmer 1972). In this test, a uniform radial pressure is applied to the walls of the bore hole over a short length. This causes a field of compressive stresses in the horizontal plane. The intensity of this stress is equal to the applied pressure at bore hole wall and becomes zero at infinite distance. Each annular ring of the soil gets proportionately deformed under the respective stress and the sum total of all such deformations shows an expansion of bore hole diameter. Since the test

length is fixed, this expansion can be measured as a volumetric deformation. In essence, the device carries out a type of load-deformation test on the adjacent soil.

2.2.1.2 Description of the Pressuremeter

The pressuremeter is shown in Fig. 2.5. This is manufactured by Associate Instrument Manufacturers (India) Pvt. Ltd. as 'Sub-soil Deformeter'. The part of the pressuremeter which goes into the bore hole is known as 'probe'. It consists of a measuring cell (12) at centre and two guard cells (13) at the ends. The measuring cell is used to measure volume changes. The guard cells serve merely to reduce the end effects. The measuring cell is connected to the bottom of measuring cell reservoir (1) by means of a hose. Another hose (running co-axially over this hose) connects the guard cells to the guard cell reservoir (4). The reservoirs are filled with water, whose level is seen in graduated volumeter (2). A set of pressure gauges (3 and 5) measures the pressures and a set of valves controls the flow of water. In this test, air compressor was used as a source of applying pressure, but it has been felt that the constant pressure CO_2 gas cylinders would be nc now desirable for conducting tests.

2.2.1.3 Test Procedure

(1) First, a bore hole of proper diameter is made to the test depth by any suitable means, preferably by hand auguring.

(2) The system is de-aired and 'probe' is attached to the co-axial hose. This way, the probe gets connected with the two reservoirs.

(3) To allow for the resistance of rubber membrane, this is calibrated by a surface test to full volumetric expansion, which gives a graph (air calibration curve) for appropriate correction in the applied pressure.

(4) After air calibration, probe is attached to the drill rod, lowered into the bore hole and held at test depth.

(5) All valves are opened so that the system attains equilibrium under atmospheric pressure. Water level of measuring cell reservoir is noted.

(6) A small step of pressure is applied to the system. This pressure causes the probe to expand and consequently water level to fall. The pressure is maintained constant and the volume readings are noted 15, 30, 60 seconds after applying the pressure.

(7) After the 60th second, the next step of pressure is applied and observations continued as before. This way, the test is carried out with equal increments of pressure till the soil fails.

(8) After performing one test at one depth , the pressure is released, probe is allowed to contract to its original size and is withdrawn to ground level.

(9) The bore hole is then extended to the next test depth and the test is repeated in the same manner.

2.2.2 Standard Penetration Test

Standard penetration test was conducted with the split spoon sampler. The split spoon was driven 45.0 cms into the ground by means of 140 lb weight (hammer) falling free height of 75.0 cm. The number of blows for each 15.0 cm was recorded. This was done by making marks at intervals of 15.0 cms on the extension rod. The total number of blows required to drive the last 30.0 cm of penetration was noted down to get the N-value for the layer penetrated. The value of 'N' has been found out in two bore holes at every one meter depth starting from 1.0 m depth to 4.0 m below ground surface.

2.2.3 Static Cone Penetration Test

This test was conducted to evaluate the resistance of soils to penetration. Semi-theoretical methods have enabled the resistances recorded by this apparatus to be translated into strength and compressibility characteristics of the soils (eg. Sanglerat 1972).

A hand operated cone penetrometer of capacity 3000 kg was used for conducting the test. The equipment consists of a truncated 60 degree cone of 10 sq.cm. base area. This cone is pushed vertically into the ground, at a constant rate. The load required to push the cone and mantle gives the total resistance to penetration. This total resistance includes the point resistance and side friction. It is possible to hold the outer mantle stationary and to advance the point independently. The load, thus measured, will give the point resistance . Subtracting the point resistance load from the total ressistance load gives the total side friction resistance . The values of point resistance and side friction were found out at 1.0 m, 2.0 m, 3.0 m and 4.0 m below ground surface.

2.2.4 Plate Load Test

This test was conducted to evaluate the bearing capacity and field values of deformation modulus. This test was performed at a depth of 2.0 m in a bore hole of 15 cm dia

with a rigid plate of 10.0 cm dia. Only one test was conducted. Special care was taken in flattening and cleaning out the bottom of auger hole. Reaction- type loading was used for this test. Three dial gauges were used to measure the settlement and were read for every load increment when the rate of settlement dropped down to 0.02 mm/minute. This type of deep plate load test has been conducted in London clay by Marsland and Randolph (1977).

2.2.5 In-Situ Permeability Test

The in-situ permeability test was conducted at a depth of 4.0 m below ground surface. Field determination of permeability is quite valuable in such alluvial soils with erratic 'kankar' layer, undisturbed samples of which are usually of very poor quality. Casagrande-type piezometer was used for this test. The complete set-up of the experiment is shown in Fig. 2.6. A brief description of the method to install the piezometer is given below.

(1) A cased hole of 2'' dia is advanced to the elevation planned for the bottom of permeable space. The first section of casing should be at least 10 ft. long. There should be a tight contact between the bottom 10 ft. of casing and surrounding soil and so in driving last 10 ft., no washing should be done.

(2) After casing pipe has been driven upto the required elevation, inside of casing is washed continuously until all cloudy water is removed.

(3) The casing is then pulled up by 2 ft.preferably by jacking andsaturated Ottawa sand is pouredin to fill bottom 2 ft. of the hole. The rate of raising the pipe and rate of pouring the sand should be such that arching of sand within the casing may not take place. During this process pipe is kept filled with water. The elevation of the top of each point must be accurately determined.

(4) The piezometer with plastic tubing attached to it is lowered into the casing pipe until it rests on the sand in the bottom of the hole.

(5) With the piezometer resting on sand the casing is pulled up an additional 2 ft. corresponding to the desired elevation of the top of the point. Saturated Ottawa sand is, then,poured in to fill the space around the point. The volume of sand needed for this operation can easily be computed and controlled.

(6) While putting the casing upto its final position, i.e. one foot above top of porous point, the open hole is back filled with more saturated sand.

(7) Enough sand is again poured into the casing to fill approximately bottom 3 ft. of casing which is then

tamped by means of 10 blows of specified hammer with a fall of 6 inch. The purpose of this sand plug is to minimize the effect of swelling pressure of overlain bentonite.

(8) Five 3 inch layers, each one of bentonite balls of about 3/8 inch dia and well tamped, provides an effective seal. In between the bentonite layers a 3/4 inch thick layer of approximately 3/8 inch dia pebbles is placed to prevent hammer from sticking. Each layer is tamped by 20 blows of specified hammer with a free fall of 6 inch.

(9) Graded sand is then poured to fill the pipe 2 ft. above sealing and tamped. Another sealing as described in (8) is placed over it.

(10) The second bentonite seal should be capped with about 3 ft. of sand and remainder of hole may be left open.

After setting, the piezometer assembly was kept continuously full of water in order to ensure a zone of saturated soil around the piezometer as the moisture content of the soil at this depth was 19.86 percent which is less than the saturation moisture content of 23 percent. Falling head method was applied for determination of permeability.

2.2.6 Oedometer Test

The consolidation tests were conducted on one horizontal and one vertical sample from the same tube sample obtained from a depth of 4.0 m.

2.2.7 Lab Determination of Permeability

A vertical sample of 10.2 cm dia and 11.7 cm height was taken from undisturbed tube sample obtained from a depth of 4.0 m. Falling head test was conducted to determine vertical permeability. Similarly, a horizontal sample of 3.8 cm dia and 7.7 cm height was also taken from the same tube sample and falling head test was conducted to get the horizontal permeability. The horizontal permeability, thus obtained, has been compared with field permeability from in-situ testing.

CHAPTER 3

TEST RESULTS AND THEIR INTERPRETATIONS

3.1 PRESSUREMETER TEST

The results of pressuremeter tests are presented in the form of plots between measuring cell pressure against the sixty seconds readings of volumeter (V_{60}) (Figs. 3.1, 3.2. and 3.3). Usually, the plot consists of three parts (Fig. 3.3). Part AB represents the expansion of the probe in the air gap. Part BC represents the stress-deformation characteristics of the soil in its pseudo-elastic phase. Part CD represents the plastic failure of the soil (THE MENARD PRESSUREMETER, 1975).

3.2 STANDARD PENETRATION TEST

The N-values, as obtained from the tests are presented in Table 3.1.

TABLE 3.1

N-VALUES WITH DEPTH

Depth (m)	Bore hole 1	Bore hole 2
	N-values	N-values
1.0	10	9
2.0	8	8
3.0	15	17
4.0	10	11

The graph between N-value against depth is shown in Fig. 3.4.

3.3 STATIC CONE PENETRATION TEST

Point resistance and average side friction, obtained from cone penetrometer, are presented in Table 3.2. Graphs between depth, cone resistance and side friction are shown in Fig. 3.5. The values of friction ratio, defined as the ratio of average side friction to cone resistance, are also shown in Table 3.2.

TABLE 3.2

VALUES OF CONE RESISTANCE, SKIN FRICTION AND FRICTION RATIO

Depth (cm)	Cone resistance (q_c) (kg/cm^2)	Average side friction(f_s) (kg/cm^2)	Friction ratio $= \frac{f_s}{q_c}$ percent
50	16	0.533	3.3
100	34	1.43	4.2
150	28	0.548	2.0
200	25	0.672	2.7
250	31	0.424	1.4
300	45	0.631	1.4
350	38	0.301	0.8
400	32	0.283	0.9

3.4 PLATE LOAD TEST

Load intensity-settlement data as obtained from the plate load test are plotted in Fig. 3.4.

3.5 IN-SITU PERMEABILITY TEST

Hvorslev (1949) has given the following formula for determination of field permeability from variable head method and the type of set up used for this test:

$$k_h = \frac{d^2 \ln\left[\frac{2mL}{D} + \sqrt{1+\left(\frac{2mL}{D}\right)^2}\right] \ln \frac{H_1}{H_2}}{8L\ (t_2-t_1)}, \text{ for } \frac{2mL}{D} < 4 \qquad \text{(Eq.3.1)}$$

For $\frac{2mL}{D} > 4$, the formula becomes

$$k_h = \frac{d^2 \ln\left(\frac{4mL}{D}\right)}{8L(t_2-t_1)} \ln \frac{H_1}{H_2} \qquad \text{(Eq. 3.2)}$$

where

D = Dia of piezometer

d = Dia of stand pipe

L = Length of piezometer

H_1 = Piezometer head for $t=t_1$

H_2 = Piezometer head for $t=t_2$

$$m = \sqrt{\frac{k_h}{k_v}}$$

Assuming $m = 1$ and using the above formula, the average horizontal permeability determined is 4.8×10^{-5} cm/sec.

3.6 CONSOLIDATION TEST

The results of the consolidation tests are presented in the form of graphs between axial strain $(\frac{dh}{h})$ and log effective vertical stress as shown in Fig. 3.7. The values of $\frac{C_c}{1+e_o}$, obtained by calculating the slope from straight line portion of the plot shown in Fig. 3.7, are presented in Table 3.3.

TABLE 3.3

Depth m	$(\frac{C_c}{1+e_o})$ Horizontal	$(\frac{C_c}{1+e_o})$ Vertical
4.0	0.0915	0.0914

The coefficient of volume compressibility(m_v) versus applied effective stress on log scale is also plotted in Fig. 3.7. It is seen that m_v attains a maximum value at a vertical effective stress of 0.8 kg/cm^2 for both vertical and horizontal samples (Fig. 3.7). This gives a value of K_o = 1 at 4.0 m depth.

3.7 LAB DETERMINATION OF PERMEABILITY

Horizontal and vertical permeabilities are determined in the lab by using the following formula for variable head

permeability test : (Lambe and Whitman, 1969).

$$k = \frac{2.3\ a\ L}{A(t_1 - t_o)} \log_{10} \frac{h_o}{h_1} \qquad \text{(Eq.3.3)}$$

where

a = Cross-sectional area of stand pipe

L = Length of soil sample in permeameter

A = Cross-sectional area of permeameter

t_o = The time when the water level in the stand pipe is at h_o

t_1 = The time when the water level in the stand pipe is at h_1

h_o, h_1 = The heads between which the permeability is determined.

The values of coefficient of permeabilities, calculated using above relationship, are given below:

(a) Horizontal permeability $k_h = 2.87 \times 10^{-5}$ cm/sec.

(b) Vertical permeability $k_v = 3.36 \times 10^{-5}$ cm/sec.

3.8 OTHER TESTS

Results of classification tests are shown in Figs. 2.2 and 2.3. Natural moisture contents were also determined. A typical bore-hole-profile for this site was established (Fig. 2.4) on the basis of these test results.

3.9 INTERPRETATION OF PRESSUREMETER TEST RESULTS

The results of pressuremeter tests conducted at 1.0 m and 3.0 m depth in different bore holes are presented in Fig. 4.2. Similarly, the results of 2.0 m and 4.0 m depths are presented in Fig. 4.3. By observing these graphs following points can be concluded:

(1) The test curves of 2.0 m and 4.0 m depths are quite close to each other and it shows that the results of the tests are reproducible at these depths.

(2) The test curves of 1.0 m and 3.0 m have a wide gap among them due to variable fill deposits at 1.0 m depth and erratic 'kankar' formation at 3.0 m depth. Thus it helps to locate variable fill deposits and erratic kankar formation.

Fig. 4.1 shows family of curves from a bore hole obtained from pressuremeter tests along with the strata met from Ground level to 12.0m deep conducted by Venkatesan (1980). The curves obtained from present investigation are similar to those obtained by Venkatesan (1980).

From the results of pressuremeter tests, the main mechanical characteristics of the soil are calculated; these are; the limit pressure p_l and the deformation modulus E.

3.9.1 Limit Pressure

The limit pressure is defined as pressure corresponding to the limiting state of failure of a soil subjected to an increasing uniform pressure on the wall of a cylindrical cavity. Thus the limit pressure is the abscissa of the asymptote to the pressuremeter curve. But more conveniently it is taken as the pressure corresponding to a volume increase V equal to the initial volume of bore hole V (i.e. $\frac{\Delta V}{V} = 1$). Since the initial volume for a standard BX probe (used in present study) is of the order of 600 cc, $\frac{\Delta V}{V} = 1$ may be assumed to occur for a reading of V=700 cc.

In the test at 3.0 m depth in bore hole 5, the volumetric increase did not attain the specified value (V = 700 cc). In such a case, to compute the limit pressure,last three readings of the plastic phase of the test curve are plotted on a p vs $\frac{1}{V}$ graph (Fig. 3.8). This plot forms a part of straight line. The point at which the extension of straight line intersects the 700 cc ordinate corresponds to limit pressure (THE MENARD PRESSUREMETER, 1975).

It is to be noted that in all these cases, p_l should be obtained by using the formula,

$$p_l = p_u - p_i + \text{water head in the tubing}$$

where p_u and p_i are shown in Fig. (3.3).

3.9.2 Deformation Modulus

Basically the pressuremeter test results are interpreted on plane strain deformation, although the test conditions do not conform perfectly to plane strain conditions except at the mid point of the probe. For a plane strain condition, the basis for interpretation of the pressuremeter test most often used is a solution for the radial deformation (Δr) of an infinitely thick elastic cylinder subjected to an internal pressure increment (Δp). (Timoshenko and Goodier, 1951)

$$\epsilon_r = \frac{\Delta r}{r_o} = \frac{(1+\nu)\Delta p}{E} \qquad \text{(Eq.3.4)}$$

where

r_o = Initial radius of hole

ν = Poisson's ratio

If V is the volume of the cavity at the instant when $\frac{\Delta p}{\Delta V}$ is measured, provided $\frac{\Delta p}{\Delta V}$ is measured in the pseudo-elastic phase of the test. Eq.3.4, when expressed in function of volume, becomes

$$E = 2\,(1+\nu)\,V\,\frac{\Delta p}{\Delta V} \qquad \text{(Eq. 3.5)}$$

In a pressuremeter test, V is the sum of two components

$$V = V_o + V_m$$

where

V_o = Initial volume of measuring cell (ie. 535 cm^3)

V_m = Mean additional volume injected (Fig. 3.3)

Eq. (3.5) , therefore, can be expressed as

$$E = K \times \frac{\Delta p}{\Delta V} \qquad \text{(Eq. 3.6)}$$

where

K = Dimensional coefficient of the probe

$= 2\ (1+\nu)(V_o + V_m)$

Short duration of loading in these tests will obtain undrained conditions ($\nu = 0.5$). So that

$$K = 3\ (V_o + V_m)$$

Taking into account the rigidity of probe, Eq. 3.6 can be expressed as

$$E = K \times \frac{\Delta p - \Delta p_i}{\Delta V} \qquad \text{(Eq. 3.7)}$$

where Δp_i is obtained from air calibration curve corresponding to Δp on the test curve (Fig. 3.3).

The values of p_l and E, calculated from pressuremeter tests, are presented in Table 3.4.

TABLE 3.4

VALUES OF LIMIT PRESSURE AND DEFORMATION MODULUS

Depth (m)	Bore hole 1			Bore hole 2			Separate bore holes		
	p_l (kg/cm^2)	E (kg/cm^2)	E/p_l	p_l (kg/cm^2)	E (kg/cm^2)	E/p_l	p_l (kg/cm^2)	E (kg/cm^2)	E/p_l
1.0	4.85	36.6	7.5	5.50	44.2	8.0	1.65	14.1	8.6
2.0	4.65	39.4	8.5	5.05	42.5	8.4	4.80	40.3	8.4
3.0	7.15	74.7	10.4	7.35	62.6	8.5	8.47	99.6	11.8
4.0	6.25	63.3	10.1	6.10	65.1	9.0	5.90	60.1	10.3

3.9.3 Ultimate Bearing Capacity

The ultimate bearing capacity of a foundation q_u is related to the limit pressure p_l of the soil by a linear function: (THE MENARD PRESSUREMETER, 1975).

$$q_u - q_o = k\,(p_l - p_o) \qquad \text{(Eq.3.8)}$$

where k is the bearing factor varying from 0.8 to 9 according to the embedment, the shape of the foundation and the nature of the soil;

q_o is the overburden pressure at the periphery of the foundation level after construction;

p_o is the horizontal earth pressure at rest but in dealing with pressuremeter, instead of p_o, the head of water in tubing over probe (h) is used generally.

Thus, Eq. 3.8 becomes

$$q_u - q_o = k\ (p_l - h\ ) \qquad \text{(Eq. 3.9)}$$

To get the allowable bearing capacity a factor of safety, F=3 is taken. Thus

$$q_{all} = \frac{q_u}{3}$$

To compute the bearing capacities, the values of k were used for an isolated footing of 1m x 1m resting on test level (THE MENARD PRESSUREMETER, 1975).

The values of k, q_u and q_{all} are presented in Table 3.5.

3.9.4 Undrained Cohesion

Gibson and Anderson (1961) have presented the following theoretical expression for cohesive soils (ϕ=0) by considering the soil as ideal elastic plastic material.

$$p_l = p_o + C_u \left[1 + \ln \left\{ \frac{E}{2C_u(1+\nu)} \right\} \right] \qquad \text{(Eq. 3.10)}$$

Taking ν=0.5 for undrained case, C_u can be calculated by trial and error by using the following equation.

$$p_l - p_o = C_u \left[1 + \ln \left(\frac{E}{3C_u} \right) \right] \qquad \text{(Eq. 3.11)}$$

In order to compute the value of C_u from Eq. 3.11, the appropriate value of undrained modulus is required.

TABLE 3.5

ULTIMATE AND ALLOWABLE BEARING CAPACITY

Depth (m)	k	q_o (kg/cm^2)	h (kg/cm^2)	Bore hole 1		Bore hole 2		Separate bore holes	
				q_u (kg/cm^2)	q_{all} (kg/cm^2)	q_u (kg/cm^2)	q_{all} (kg/cm^2)	q_u (kg/cm^2)	q_{all} (kg/cm^2)
1.0	1.50	0.187	0.2	7.16	2.39	8.14	2.71	2.36	0.79
2.0	1.78	0.363	0.3	8.11	2.70	8.82	2.94	8.37	2.79
3.0	1.83	0.567	0.4	12.92	4.31	13.29	4.43	15.34	5.11
4.0	1.83	0.763	0.5	11.29	3.76	11. 01	3.67	10.65	3.55

However, Marsland and Randolph (1977) have proposed that to get the representative value of C_u in the undisturbed material, E should represent the overall equivalent value for undisturbed material. Generally the deformation modulus measured in pressuremeter is likely to be lower than the in-situ value. The pressuremeter E values may be corrected by comparing the results with the plate load tests. The value of E obtained from plate load test at 2.0m depth is 72.5 kg/cm^2 while the average value of E determined by the pressuremeter test at 2.0 m depth is 40.77 kg/cm^2. So a ratio of 1.8 exists between plate load modulus and pressuremeter modulus. Thus a representative deformation modulus at other depths may be obtained by using this ratio. Using these E values in Eq.3.11, the values of C_u obtained are presented in Table 3.6.

TABLE 3.6

UNDRAINED COMPRESSIVE STRENGTH WITH DEPTH

Depth (m)	Bore hole 1	Bore hole 2	Separate bore holes
	C_u (kg/cm^2)	C_u (kg/cm^2)	C_u (kg/cm^2)
1.0	1.20	1.35	0.35
2.0	1.05	1.13	1.15
3.0	1.45	1.65	1.75
4.0	1.25	1.25	1.15

3.10 INTERPRETATION OF PLATE LOAD TEST DATA

Kee's method has been applied to interpret the data. According to this method, the equation of load-settlement characteristic is hyperbolic in nature and the following equation can be written for it.

$$y = \frac{x}{a + bx}$$

This equation can also be expressed as

$$\frac{x}{y} = a + bx$$

where, a and b are constants and can be determined by plotting x/y versus x and fitting a straight line.

Using this method, a graph between $\frac{\text{settlement}}{\text{load intensity}}$ and settlement is plotted in Fig. 3.9. The values of a and b are shown in this Figure.

The 1/a value gives initial modulus of subgrade reaction $(k_s)_i$ and 1/b gives ultimate bearing capacity q_u.

The following results were obtained from plate load test data using this interpretation.

(a) Initial modulus of subgrade reaction $(k_s)_i = 21.505$ kg/cm^3

(b) Ultimate bearing capacity q_u = 19.04 kg/cm^2

(c) Allowable bearing capacity $= \frac{q_u}{3} = 6.3$ kg/cm^2 .

3.10.1 Deformation Modulus

The theory of elasticity gives the following expression for displacement ρ under a circular rigid plate of dia B with a load intensity of q.

$$\rho = \frac{q.B.(1-\nu^2)}{E} \times \frac{\pi}{4} \qquad \text{(Eq. 3.12)}$$

where

$k_s = \rho/q$ and

$\nu = 0.5$ (undrained condition)

Burland and Marsland (1971) suggested that, for D/B > 6, Eq.3.12 becomes

$$\rho = \frac{q.B.\ (1-\nu^2)}{E} \times \frac{\pi}{4} \times f(z)$$

where f(z) is depth factor equal to 0.85(for D/B > 6) which takes in to account the stiffening effect of soil above test level.

Thus, E_u can be computed using following relationship

$$E_u = 0.59 \times k_s \times B \times 0.85 \qquad \text{(Eq. 3.13)}$$

Again, from theory of elasticity

$$E_u = \frac{3E'}{2(1+\nu')}$$

ν', for these soils, is taken as 0.28 (Srivastava, 1977). Hence

$$E' = 0.85\ E_u$$

Secant modulus was obtained by putting the value of k_s found out at the stress level equal to one third of ultimate stress (allowable stress level) in Eq.3.13.

The following results were obtained from the plate load test data.

(a) Initial undrained modulus = 107.78 kg/cm^2

(b) Secant undrained modulus = 72.05 kg/cm^2

(c) Drained modulus = 61.63 kg/cm^2.

CHAPTER 4

COMPARISON AND CORRELATION

4.1 GENERAL

In this chapter, the characteristics obtained from the tests performed have been compared with those available in literature for similar soils. Correlations between various test results have been also obtained and the correlation factors thus obtained have been compared with those available in literature.

4.2 DEFORMATION MODULUS AND LIMIT PRESSURE OBTAINED FROM PRESSUREMETER TESTS

From the tests conducted at 1.0m depth in bore hole 3 (Fig. 3.3), the values of E and p_L were found to be 14.1 kg/cm^2 and 1.65 kg/cm^2 respectively. At this point the N-value obtained was very low (N=3) which indicates a recent fill. The values of E and p_L obtained from above test are quite comparable with **Venkatesan's** (1980) findings at 2.0m depth for loose silty sand overlain by recent fill (Fig. 4.1). Due to erratic distribution of "Kankar", the values of E and p_l at 3.0m depth were found to vary over a wide range (Fig. 4.2). At the depths of 2.0m and 4.0m (Fig. 4.3), the values of E and p_l were found to be quite

consistent and they compare favourably with the typical values of silty soils reported elsewhere (THE MENARD PRESSUREMETER, 1975).

4.3 THE $\frac{E}{p_1}$ RATIO

It has been reported (THE MENARD PRESSUREMETER , 1975) that $\frac{E}{p_1}$ ratio is a characteristic of the soil type; the higher values of $\frac{E}{p_1}$ (12 to 30) are encountered in over-consolidated soils and the low values (5to 8) are more prevalent in alluvial soils. The values of $\frac{E}{p_1}$ obtained from this investigation vary from 8 to 12, which is comparable with the values applicable for lightly overconsolidated alluvial soils.

4.4 VARIATION OF E and p_1 WITH DEPTH

Variations of E and p_1 with depth for campus soil, along with the values obtained by Venkatesan (1980),are shown in Fig. 4.5. The trend of the curves for campus soil matches well with the trend of the plots indicating N-values Vs. depth in Fig. 3.4 and q_c Vs. depth in Fig. 3.5.

4.5 PRESSUREMETER MODULUS AND PLATE-LOAD MODULUS

Marsland and Randolph (1977) indicated that the modulus determined from pressure meter test is lower than that from plate-load test. The same was observed by comparing

the average pressuremeter modulus obtained at 2.0m (i.e 40.73 kg/cm^2) with the plate load test modulus at 2.0m (i.e. 72.5 kg/cm^2). The ratio of E_u (plate) to E_u (pressure meter) is 1.8.

4.6 BEARING CAPACITY OBTAINED FROM PRESSURE METER TEST AND FROM OTHER TESTS

So far, the plate-load test has been used to find out the bearing capacity of soil at shallow depth (i.e. 1.0 m). The bearing capacity of the saturated soil was found to be 1.4 kg/cm^2 (Jain 1978) at 1.0m depth for a strip footing of 1.0m width. The value obtained from pressuremeter test, conducted at the same depth, is 2.55 kg/cm^2 at the natural moisture content of 8.8%. Jain (1978) has shown that the allowable bearing capacity computed from plate load tests varies with moisture content and the values for fully submerged case are roughly 50% of the results obtained for partially saturated condition. The findings of Venkatesan (1980) with this pressuremeter (Fig. 4.4) also indicate that p_1 decreases as the water table is approached (by about a factor of 1.5). It is suggested that if the bearing capacity obtained from pressure meter test for partially saturated condition is reduced by a factor of 1.5 to 2.0, the value of allowable bearing capacity agrees well with that obtained from other tests such as plate load, static

cone and c', ϕ' obtained from undisturbed block samples. The ratio of 1.03 between corrected q_{all} from pressuremeter ($\frac{2.55}{1.75}$ = 1.45) and q_{all} of 1.4 kg/cm^2 recommended by Yudhbir et al. (1979) agrees well with similar findings of Marsland and Randolph (1977) for London clay,

4.7 VARIATION OF VERTICAL EFFECTIVE STRESS DUE TO WATER TABLE FLUCTUATIONS

It has been demonstrated (Yudhbir et.al., 1979) that the soil deposits at site B on I.I.T. Kanpur campus have experienced seasonal fluctuations of water table from 0.8 m below G.L upto greater than 5 m depth below G.L. Based on his experience in Kanpur area and other sites on campus Dr. Yudhbir has suggested that water table fluctuates from 2m below G.L. to about 6m below G.L.

In order to check these water table fluctuations vertical effective stress variations with depth are computed by the following relationship proposed by Parry (1971):

$$\sigma_{vm'} = \gamma z_p - \gamma_w (z_p - z_m) \quad \text{(Eq. 4.1)}$$

$$\sigma_{vo'} = \gamma z_p - \gamma_w (z_p - z_o) \quad \text{(Eq. 4.2)}$$

where

$\sigma_{vm'}$ = effective vertical stress at any depth z_p below ground surface

z_m = depth of water table at its lowest position ,

z_o = depth of water table when it is nearest to the ground surface.

Fig. 4.5 gives variation of σ'_{vo}, σ'_{vm} and OCR with depth as computed from Eqs. 4.1 and 4.2, using z_o=0.8m and z_m=5.0m and z_o=2.0m and z_m=6.0m values as suggested above. Also shown are the values of OCR as worked out from oedometer test data on undisturbed samples for both sites A and B. Reasonable agreement between oedometer results and computations based on Eqs. 4.1 and 4.2 lends support to the assumed fluctuations of water-table in these deposits in this area. These values of σ'_{vo}, thus computed and shown in Fig. 4.5, can be used for estimation of variation of undrained strength with depth on the basis of effective stress parameters.

4.7.1 Undrained Strength Profile-Comparison of Pressuremeter Data with Other Test Values

Fig. 4.8 shows values of C_u obtained from pressuremeter, unconfined compression tests and analysis of plate load test. Also shown is the variation of long term strength as calculated from the following equation (Leonard, 1962) using σ'_{vo} values from Eq. 4.2.

$$\frac{C_u}{\sigma_{vo}'} = \frac{\frac{C'}{\sigma_{vo}'}\, \text{Cos}\, \phi' + \text{Sin}\, \phi' \left[K_o + A_f (1-K_o)\right]}{1+2\, (A_f - 1)\, \text{Sin}\, \phi'} \qquad \text{(Eq. 4.3)}$$

where

C' = Drained cohesion

ϕ' = Friction angle based on effective stresse

K_o = Coefficient of earth pressure at rest

A_f = Pore pressure parameter at failure.

Using $C' = 0.27$ kg/cm^2 , $\phi' = 26.8^o$, K_o =1.0, and A_f=0.4 as proposed by Yudhbir et.al. (1979); Eqn. 4.3 yields:

$$\frac{C_u}{\sigma_{vo}'} = \frac{0.98\, C'}{\sigma_{vo}'} + 0.49.$$

The two vertical lines in Fig. 4.6 show the vertical and horizontal values of C_u obtained from unconfined compression tests for site B (Srivastava, 1977). Based on all the available test results, a strength profile a b c d is suggested for campus soil which is consistent with the fact that in overconsolidated soils the undrained strength either decreases with depth as in part a-b, or remains constant with depth as in part b-c. At point c (depth=14 m) where OCR is 1.25, the soil tends to behave like a normally consolidated material and undrained strength increases with depth (part c-d).

It is to be noted that the strength profile a b c d (Fig. 4.6) has been proposed on the basis of test results at

shallow depths (maximum upto 4m). It is suggested that unconfined, plate load and pressuremeter tests should be conducted upto 10-15 m depth inorder to confirm or modify the proposed strength profile.

The pressuremeter test values of C_u lie close to the range of C_u values for horizontal samples, except at 3.0m depth where a 'kankar' layer was encountered. This observation seems to be consistent with the fact that, in case of a pressuremeter test, a horizontal stress field is applied on the adjacent soil and as much the C_u values inferred from the pressuremeter data are more representative of the undrained strength in the horizontal direction.

4.8 CORRELATION BETWEEN LIMIT PRESSURE AND POINT RESISTANCE

The average values of limit pressure and point resistance for each test depth are presented in Table 4.1.

TABLE 4.1

CURRELATION FACTOR BETWEEN LIMIT PRESSURE AND POINT RESISTANCE

Depth (m)	Limit pressure p_l (kg/cm^2)	Point resistance q_c(kg/cm^2)	Correlation Factor $n = \frac{q_c}{p_l}$
1.0	5.18	34.0	6.5
2.0	4.83	25.0	5.3
3.0	7.66	45.0	5.8
4.0	6.08	32.0	5.3

The values of correlation factor are found to vary from 5.3 to 6.5. It has been reported (THE MENARD PRESSUREMETER, 1975) that the ratio q_c/p_l is constant for a given geological layer but varies with the grain size distribution of the soil and its water content. For silt, this ratio varies in between 5 and 6 which agrees well with the values obtained.

4.9 CORRELATION BETWEEN SPT AND SCPT DATA

The results of SPT and SCPT are presented in Table 4.2.

TABLE 2.4

CORRELATION FACTOR BETWEEN N-VALUES AND POINT RESISTANCE

Depth (m)	q_c (kg/cm^2)	N-value (average)	Correlation factor $n = \frac{q_c}{N}$
1.0	34	10	3.4
2.0	25	8	3.1
3.0	45	16	2.8
4.0	32	11	2.9

De Alenear Velloso (1959) has given n value to be 3.5 for clays, silty clay and clayey silt. Engineers of Franki Pile

indicated n=3 for silty clay and similar deposits. In India n=6 for sandy soils and n=2 for clayey soils have been observed. So, the values of correlation factor n agree well with the earlier obtained results.

4.10 CORRELATION BETWEEN q_c AND C_u (PRESSUREMETER)

The undrained cohesion values from pressuremeter tests and cone resistance values from static cone penetration tests are presented in Table 4.3.

TABLE 4.3

CORRELATION FACTOR BETWEEN CONE RESISTANCE AND UNDRAINED COHESION

Depth (m)	C_u(pressuremeter) (kg/cm^2)	q_c (kg/cm^2)	Correlation factor $\beta = \frac{q_c}{C_u}$
1.0	1.28	34.0	26.5
2.0	1.11	25.0	22.5
3.0	1.62	45.0	27.7
4.0	1.22	32.0	26.2

Sanglerat (1972) has suggested that for the kind of penetrometer used for present study, undrained strength is approximately $q_c/20$. Schmertmann (1975) has suggested that, in cohesive soils , C_u may be obtained as;

$$C_u = \frac{q_c - \gamma D}{N_f}$$

where

γD = Total overburden pressure at cone tip

N_f = Bearing capacity factor ranging from about 5 to 70 but probably in the range of 10 for electrical cones and 16 for mechanical cones and clays with OCR < 2 and I_p > 16.

The soil investigated was found to be lightly overconsolidated with a overconsolidation ratio of 1.5 to 2.3 and as the mechanical cone was used to obtain q_c, the values of correlation factor β are comparable with the suggested values of Schmertmann (1975).

4.11 COMPARISON BETWEEN SCPT DATA AND BEARING CAPACITY FROM PRESSUREMETER TEST

San-glerat (1972) has suggested that for the design of shallow footings with an embedment of 1.0 m and resting on dense sand, the allowable bearing capacity is equal to one-tenth of point resistance of static penetrometer , i.e.

$$q_{all} = \frac{q_c}{10}$$

He made another suggestion that above relation can also be used when stiff clays, sandy clays or silty sands are encountered.

Using above relationship, the bearing capacity values for each layer penetrated are given in Table 4.3.

TABLE 4.3

Depth (m)	Point resistance q_c (kg/cm^2)	Allowable bearing capacity (cone) (kg/cm^2)	Allowable bearing capacity (pressuremeter) (kg/cm^2)
1.0	34.0	3.4	2.55
2.0	25.0	2.5	2.84
3.0	45.0	4.5	4.62
4.0	32.0	3.2	3.66

These values of bearing capacity (cone) are quite comparable with the values of pressuremeter.

4.12 COMPARISON OF CONE RESISTANCE AND FRICTION RATIO VALUES

Sanglerat (1972) has classified the soils based on cone resistance and friction ratio. The soil encountered in present study (clayey silt) should have $q_c > 30$ bar, 2 percent < FR < 4 percent, which agree well with the experimental findings. It is to be noted that it is not possible to determine the accurate values of side friction in lower layers because with movement of additional tubes penetrating the lower layers, the soil in contact with the mantle tube in upper layers is remoulded to a greater degree. Due to above reason the friction ratio observed below 2.0 m was less than 2 percent.

To get the exact value of friction ratio, special friction sleeve should be used to find out the local skin friction,

4.13 COMPARISON OF PLATE LOAD TEST RESULTS WITH PRESSUREMETER TEST RESULTS

The allowable bearing capacity was obtained as 6.3 kg/cm^2 from the plate load test conducted at 2.0 m depth with a round plate of 10 cm dia. From Jain's (1978) findings for effect of plate size on bearing capacity (for saturated condition), the q_{all} value for this size of plate is double the value of q_{all} for usual plate size of 30cm x 30cm. Therefore, the value of q_{all} obtained can be halved to take into account the plate size effect. Then the value of q_{all} becomes 3.15 kg/cm^2 which is comparable with the value of 2.84 kg/cm^2 obtained from pressuremeter test at this depth.

4.14 COMPARISON OF PERMEABILITY TESTS

Lambe and Whitman (1969)have given a chart based on permeability test data for soil classification. For most of the silts the value of permeability is of the order of 10^{-5} cm/sec. A value of 4.8×10^{-5} cm/sec. is the representative value of field permeability for these soils.

The ratio of horizontal permeability to vertical permeability is approximately equal to unity. The ratio of field to lab permeability for this soil is 1.7. This finding

is consistent with those reported by Raymond and Azzouz (1969) and others.

Attempt has been made to plot the consolidation test data as dial gauge reading versus square root of time or logarithm of time (Fig. 4.7) for computing the coefficient of consolidation C_v and then computing coefficient of permeability by back figuring. As will be seen both the $\sqrt{t}$ and log t plots do not have the characteristic shape needed to compute t_{90} or t_{50} from dial readings. Most of the change in dial readings during a load increment has taken place even before the 0.25 minute reading. Thus for these soils it is not possible to back figure k values from consolidation test.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

The following are the conclusions based on pressuremeter test results reported in this study.

(1) The results obtained from tests conducted at different depths in a sequence of first conducting the test at that depth and then extending the bore hole to the next test depth and results from tests conducted in seperate bore holes for each depth do not differ much and so effect of one test on the next test, conducted in the same bore hole, has not been observed. In these deposits the tests may thus be conducted in the same bore hole.

(2) E and p_l profiles with depth obtained from present investigation compare well with the profiles for silty sand and clayey silt reported by Venkatesan (1980) with the same instrument.

(3) For homogeneous deposits the data obtained from three tests at each depth are almost identical (Fig. 4.3), whereas in erratic profile (eg. 'kankar' layer or random fill) the repeated tests show considerable scatter (Fig. 4.2). Thus the pressuremeter test can be relied upon to bring out any erratic profile with depth.

(4) It reasonably predicts the bearing capacity taking into account the type of foundation and depth of embedment. The results for an isolated footing of 1m x 1m resting at test levels agree well with the predictions of bearing capacity from results of static cone penetration test.

(5) The trend of the curves E and p_1 with depth (Fig.4.4) compares well with the trend of the curves of SPT and SCPT test results (Figs. 3.4 and 3.5).

(6) The values of deformation modulus from pressuremeter tests are appreciably lower than those determined from in-situ plate load tests, which is consistent with similar findings reported in literature (Marsland and Randolph, 1977).

(7) Following correlations have been established between the pressuremeter test results and other test results which agree well with those available in literature (given inside the bracket).

(a) $\frac{E}{p_1}$ from pressuremeter tests varies from 8 to 12 (5-8 in alluvial soils and 12-30 in overconsolidated soils)

(b) $\frac{q_c}{p_1}$ varies from 5 to 6 (·5-6 For silts)

(c) $\frac{q_c}{N}$ Varies around 3 (3 for clayey silt and similar deposits)

(d) $\frac{q_c}{C_u}$ varies from 22 to 28 (20 for cohesive soils)

Results of this investigation indicate that the soil deformeter variety of pressuremeter manufactured by AIMIL works satisfactorily for alluvial deposits.

The following are the conclusions based on other test results reported in this study.

(8) A value of horizontal field permeability of 4.8×10^{-5} cm/sec. has been computed from the variable head field permeability test for these silty deposits and a ratio of 1.7 has been obtained between the $(k_h)_{field}$ and $(k_h)_{lab}$.

(9) It is not possible to back figure k values from consolidation test results for these soils as most of the deformation takes place rather quickly and the conventional $\sqrt{t}$ or log t fitting method is not applicable.

(10) These deposits are shown to be lightly over-consolidated due to water-table fluctuations from 2.0m to 6.0m below G.L. for site A and 0.8mto 5.0m for site B on I.I.T.Kanpur Campus. The OCR values estimated from these water table fluctuations agree well with those computed from the oedometer tests on undisturbed samples (Fig.4.5).

(11) On the basis of available results and effective stresses consistent with water-table fluctuations, undrained strength profile (for short term, ϕ=0 and long term conditions) is proposed (Fig. 4.6).

(12) It is recommended that deep bore hole plate load tests and pressuremeter tests should be conducted upto 10.0 m depth to verify or modify the proposed strength profile.

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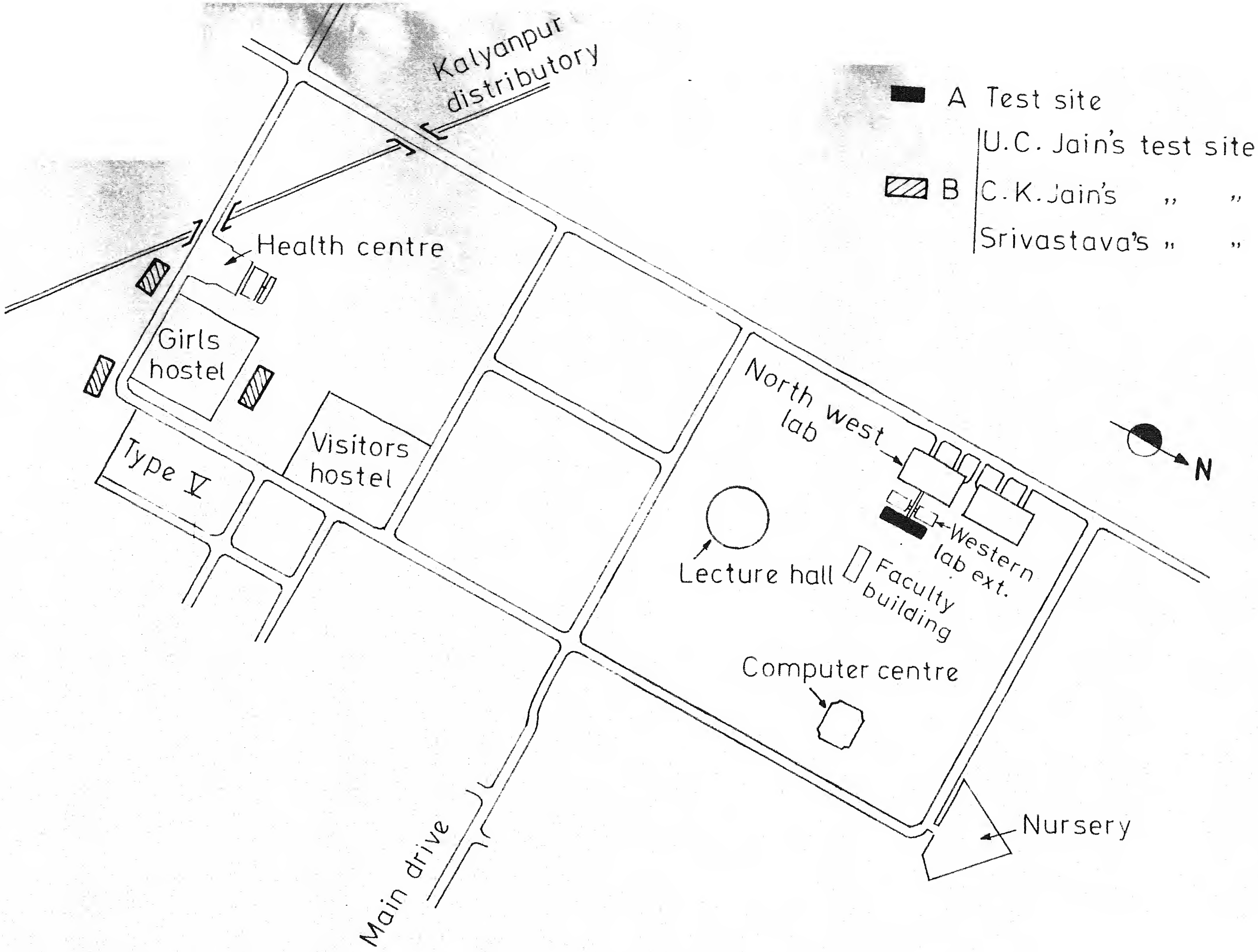


FIG 2·1 LOCATION OF TEST SITE IN CAMPUS

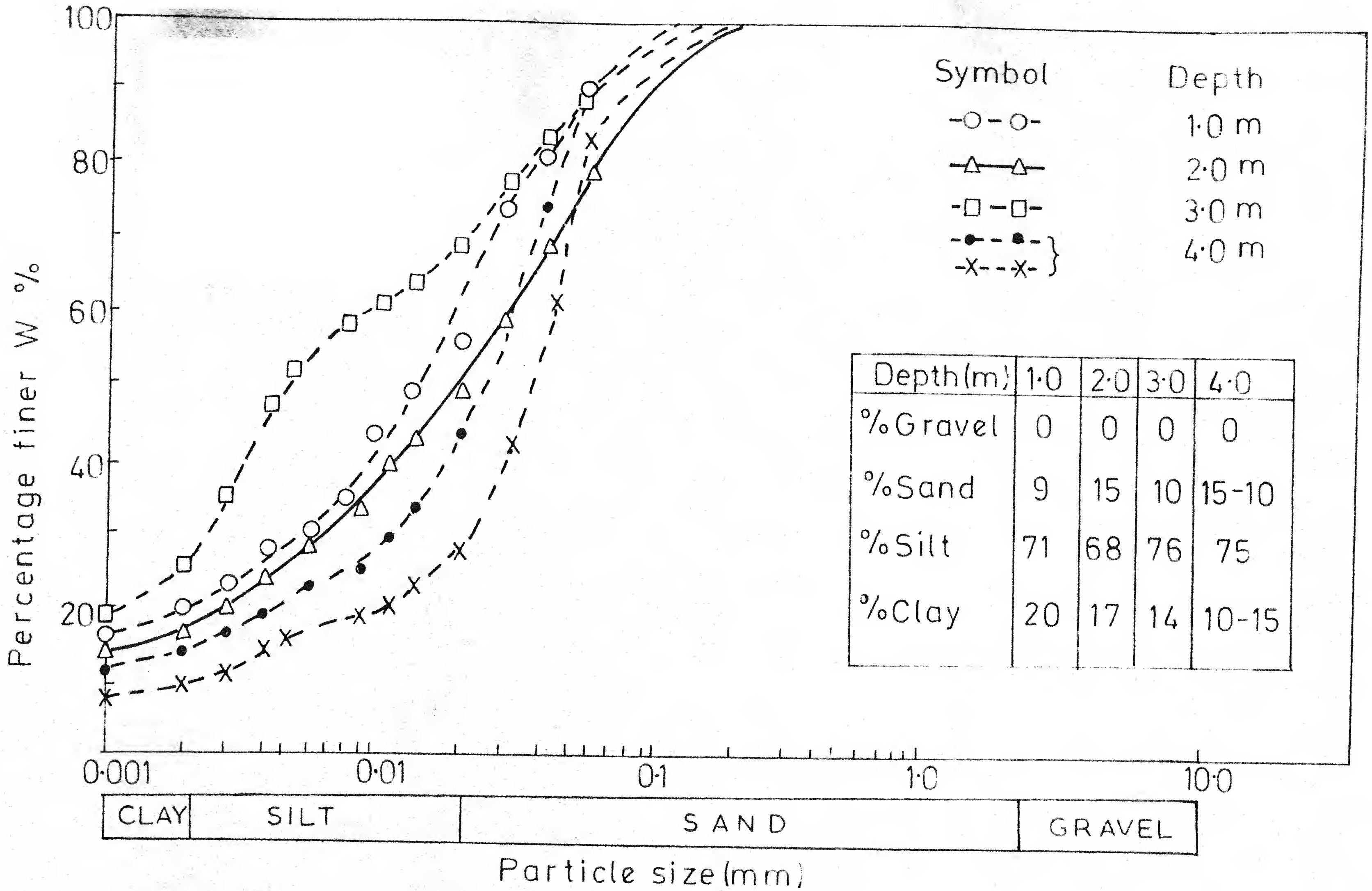


FIG. 2·2 PARTICLE SIZE DISTRIBUTION CURVE

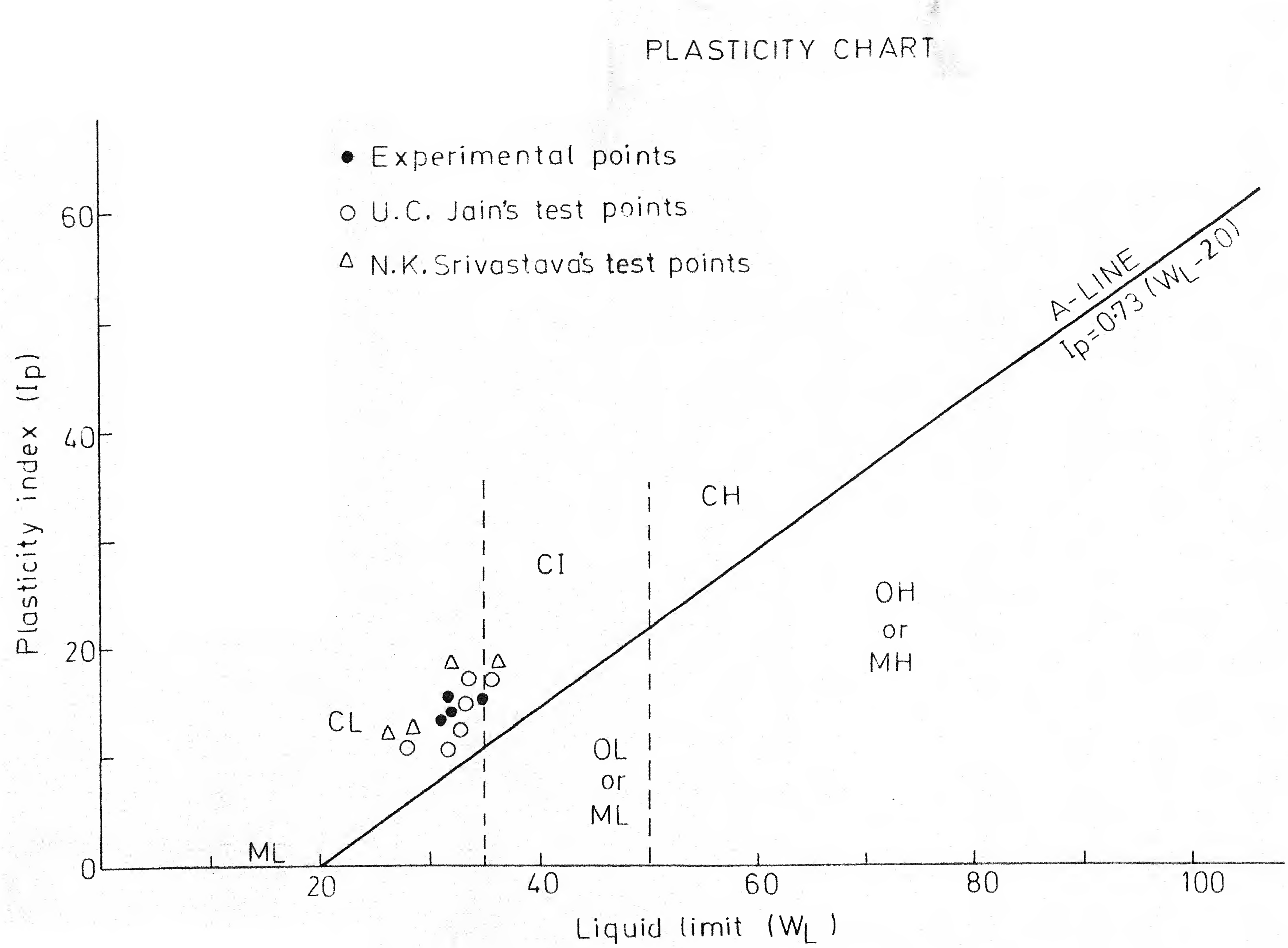
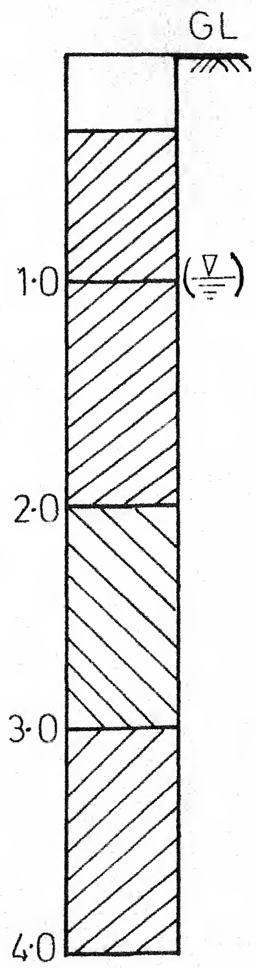
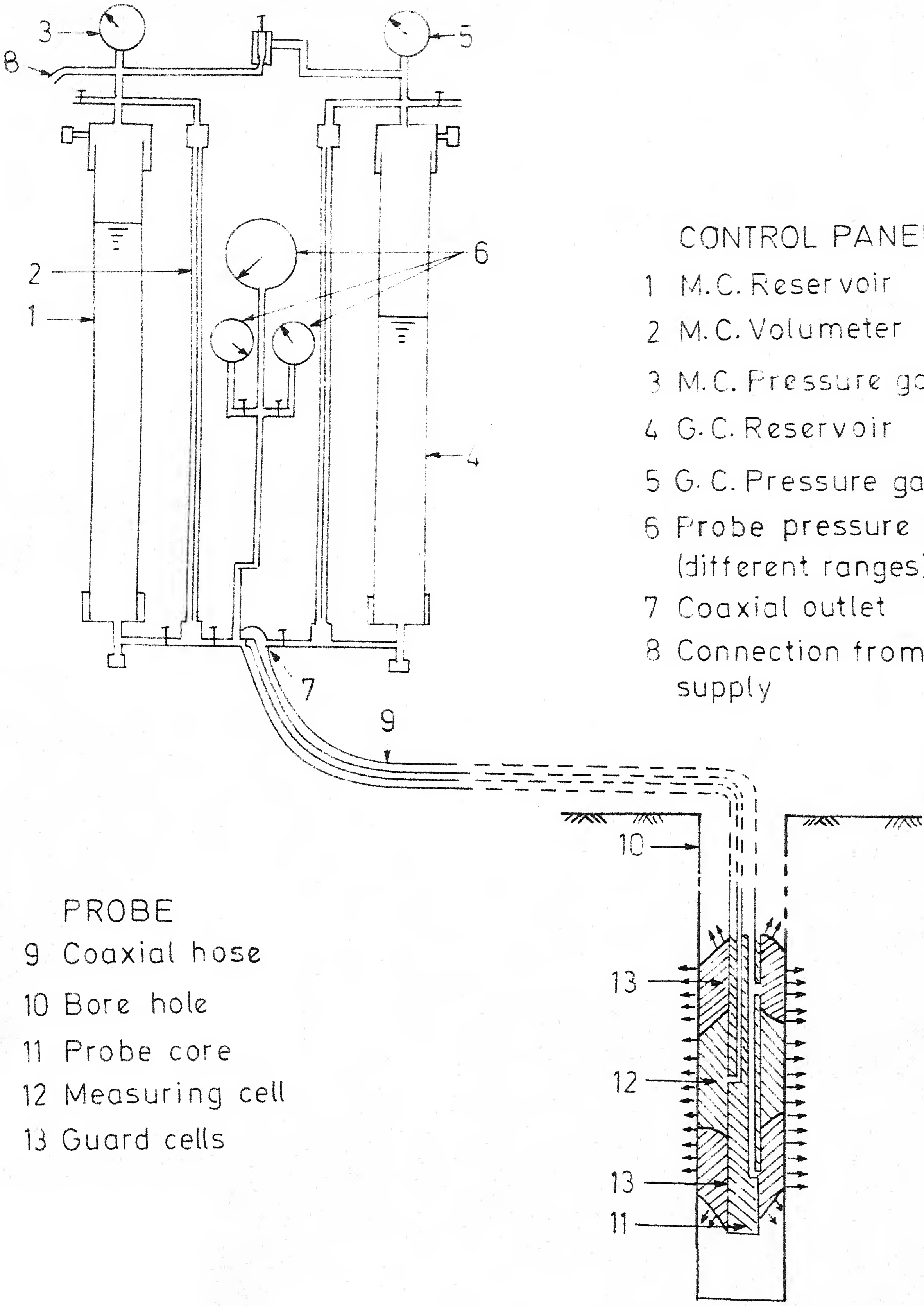


FIG. 2·3 CLASSIFICATION OF SOILS



Description of strata	Soil classification	W_n %	W_L	W_p	γ_t gm/cc	Fine sand ·2–·06 mm %	Silt ·06–·002 mm %	Clay < ·002 mm %
Recent fill								
Clayey silt with fine sand	CL (CL)	(17·8) 8·8	(33·5) 32·1	(16·3) 18·1	1·87	(17) 9	(66) 71	(17) 20
Clayey silt with fine sand	CL (CI)	(18·7) 12·0	(34·6) 31·5	(17·7) 17·7	1·76	(8) 15	(68) 68	(24) 17
Clayey silt with fine sand & kankar	CI (CL+CI)	(19·9) 14·8	(34·6) 35·0	(18·8) 19·8	2·04	(10) 10	(58) 76	(32) 14
Clayey silt with fine sand	CL (CL)	(24·6) 18·8	(31·5) 31·8	(20·3) 15·6	1·96	(16) 10	(74) 75	(10) 15

FIG. 2·4 A BORE HOLE PROFILE



CONTROL PANEL

1 M.C. Reservoir
2 M.C. Volumeter
3 M.C. Pressure gauge
4 G.C. Reservoir
5 G.C. Pressure gauge
6 Probe pressure gauges (different ranges)
7 Coaxial outlet
8 Connection from pressure supply

PROBE

9 Coaxial hose
10 Bore hole
11 Probe core
12 Measuring cell
13 Guard cells

FIG. 2·5 PRESSUREMETER

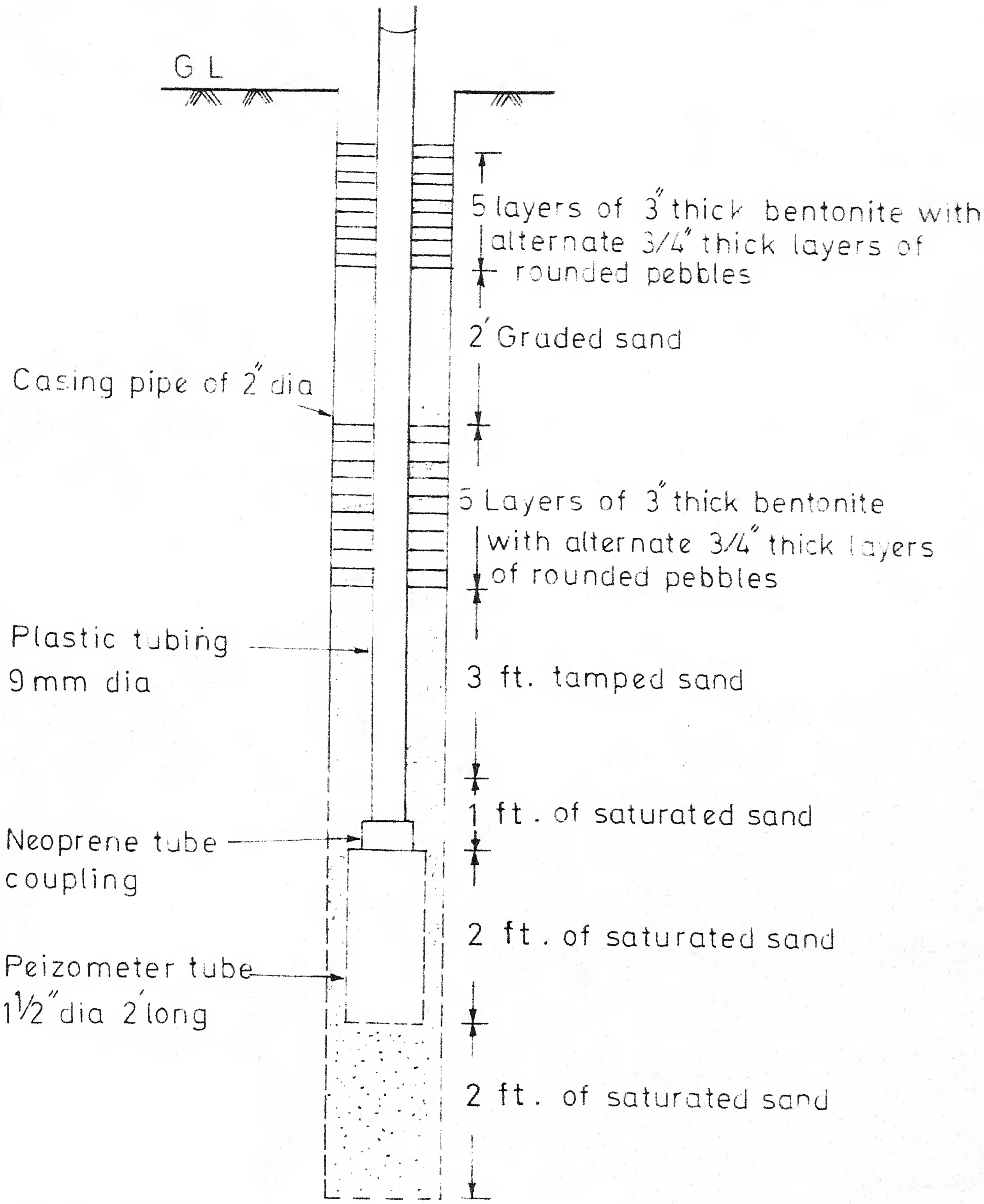


FIG. 2·6 COMPLETE SETUP OF FIELD PERMEABILITY TEST

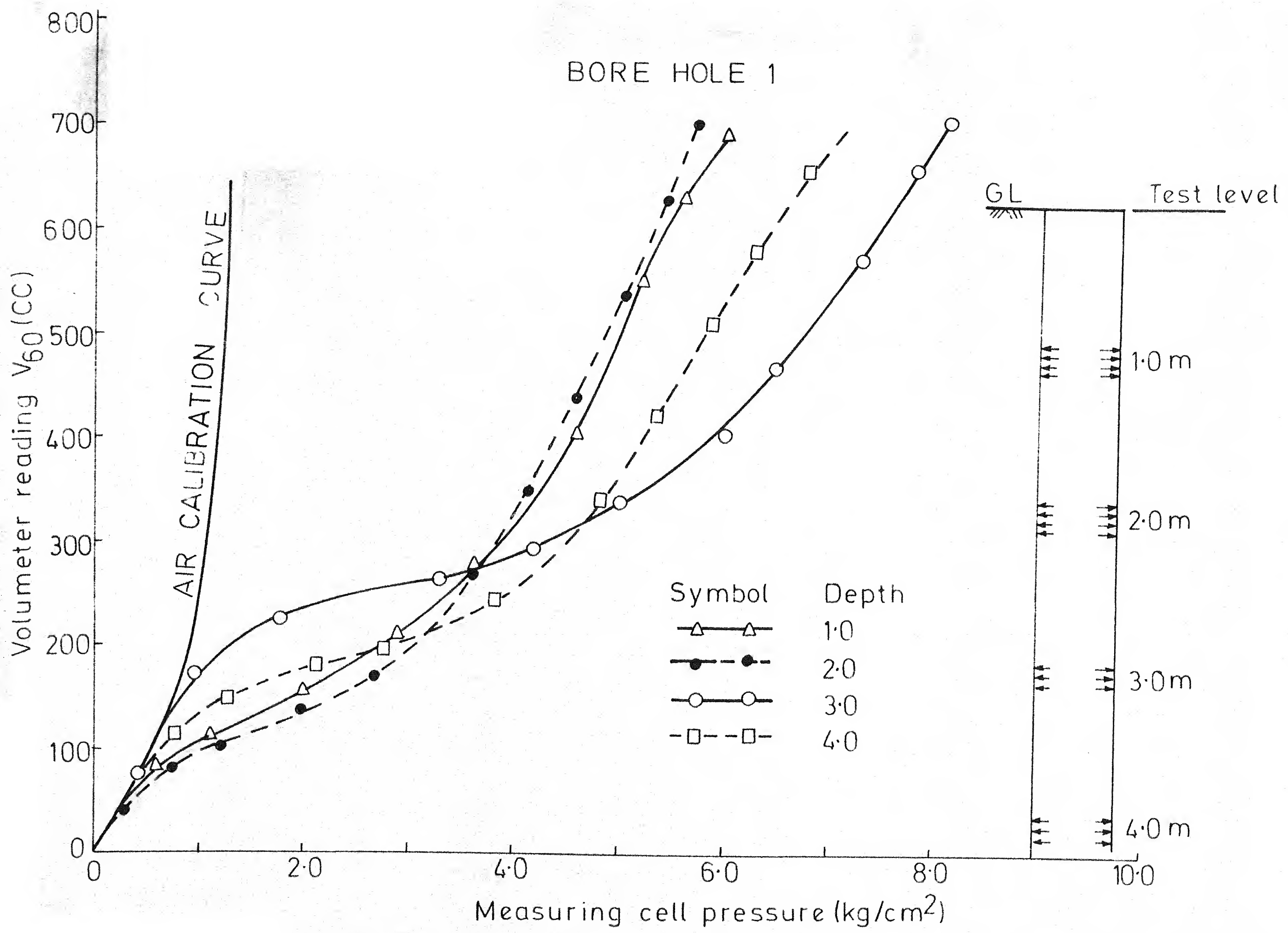


FIG. 3·1 RESULTS OF PRESSUREMETER TEST

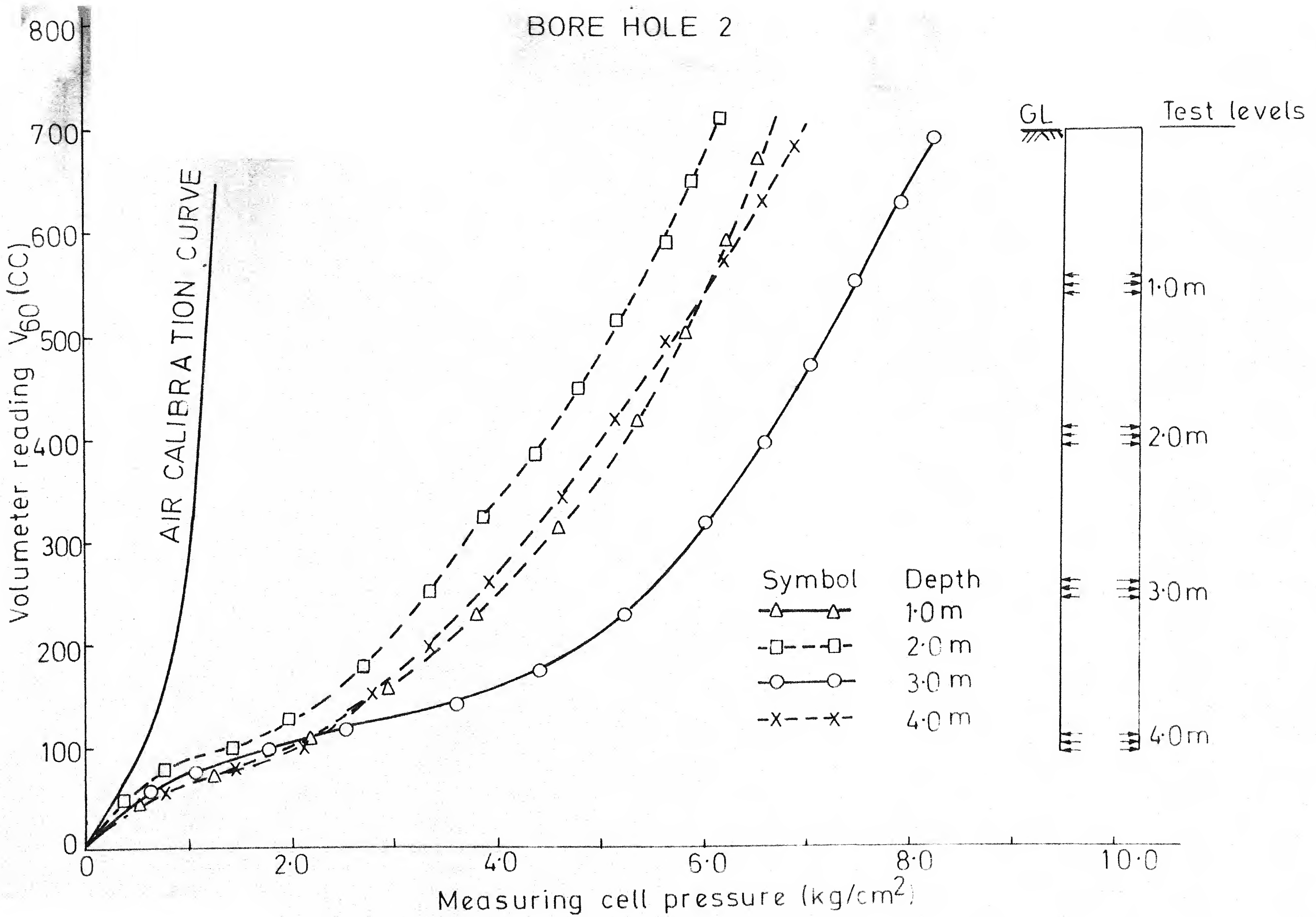


FIG. 3·2 RESULTS OF PRESSUREMETER TEST

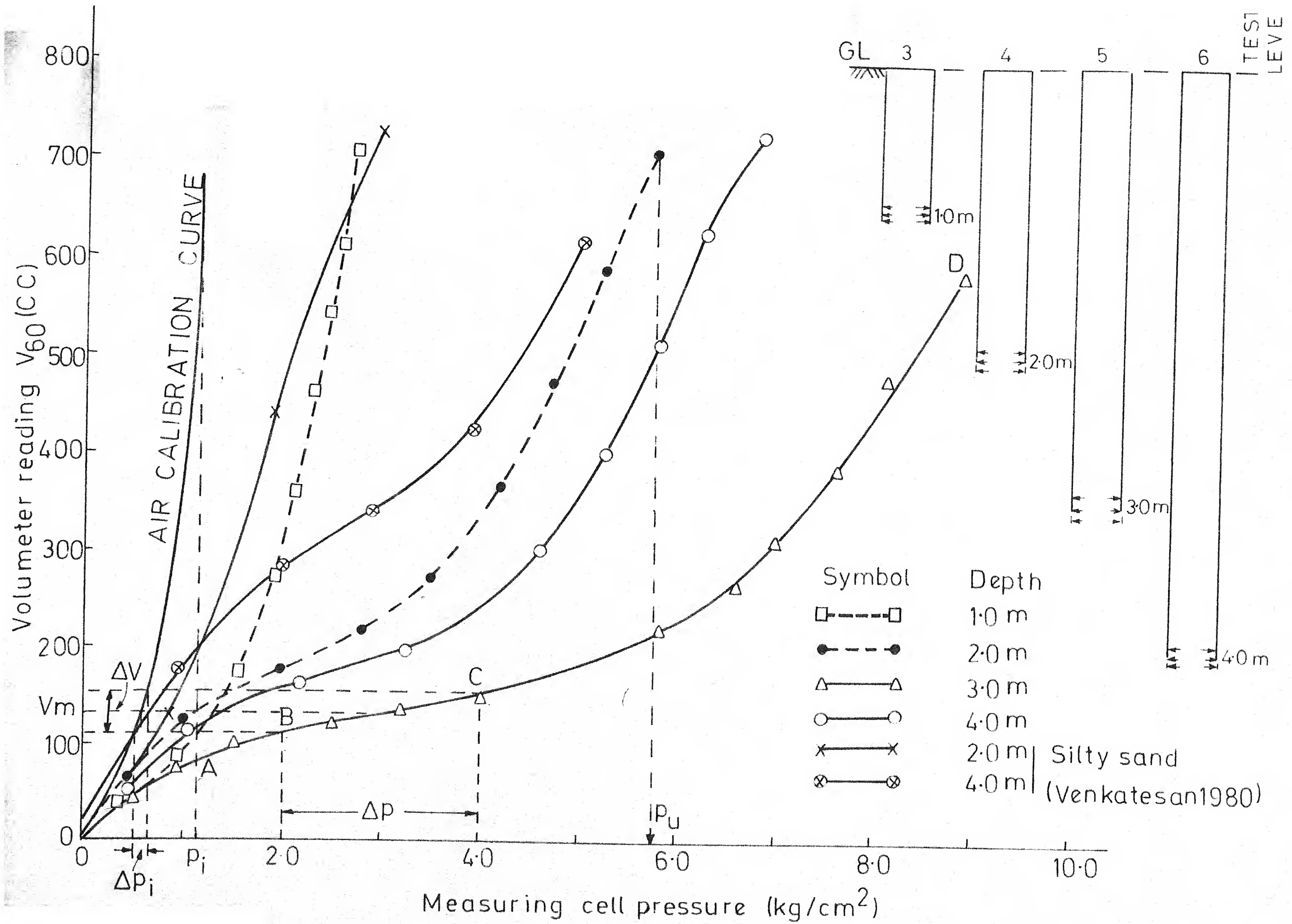


FIG. 3·3 RESULTS OF PRESSUREMETER TEST

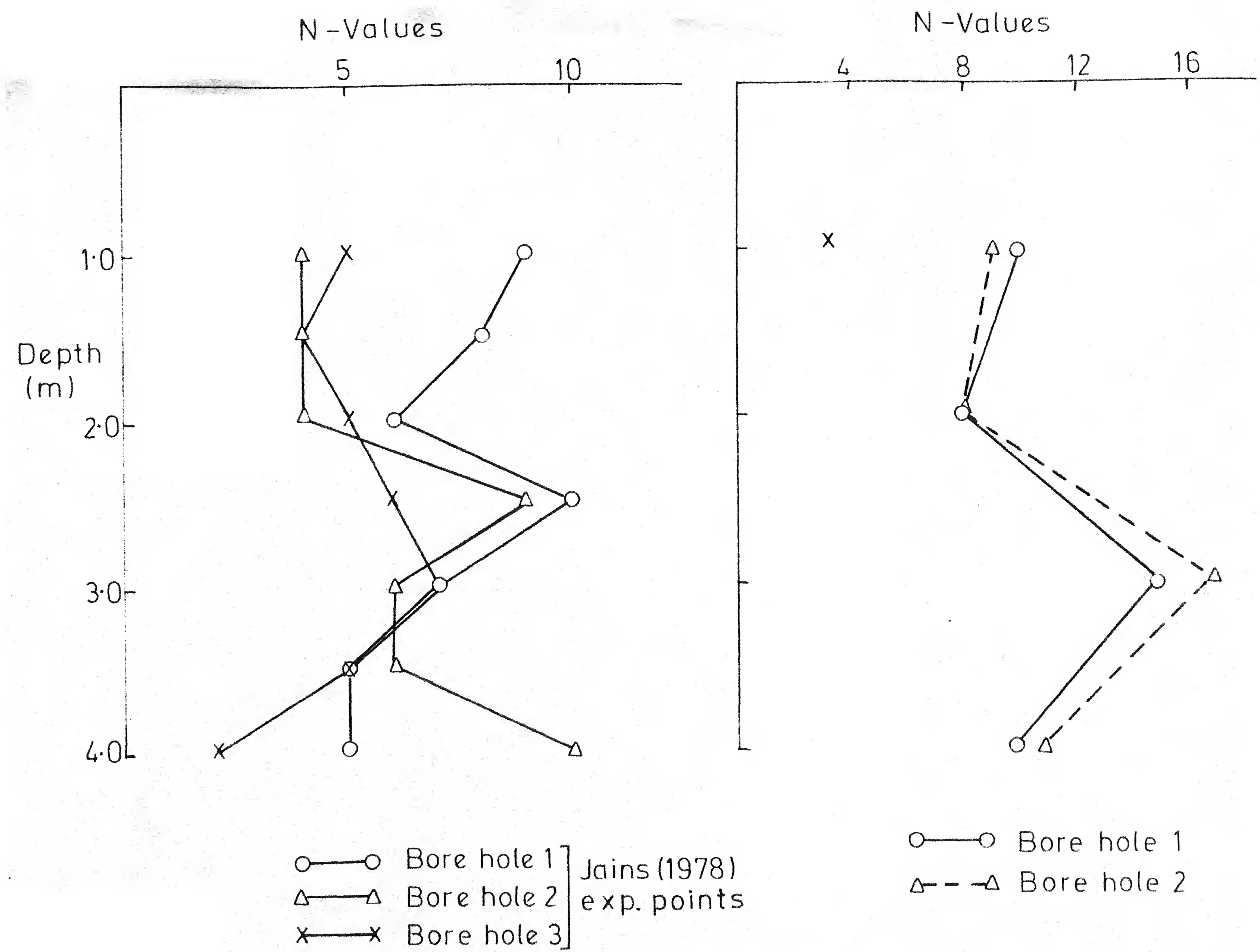


FIG. 3·4 RESULTS OF STANDARD PENETRATION TEST

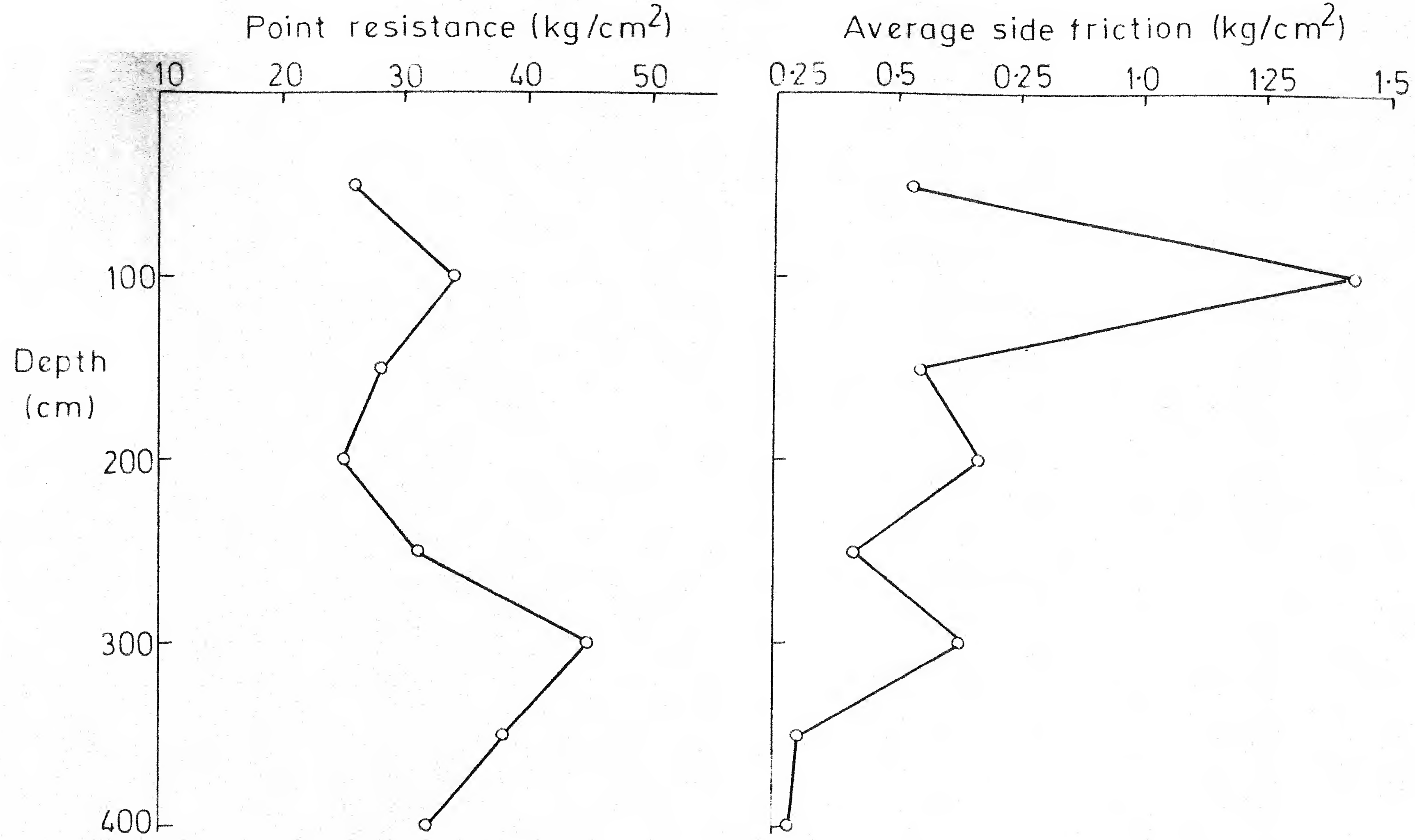


FIG. 3·5 RESULTS OF STATIC CONE PENETRATION TEST

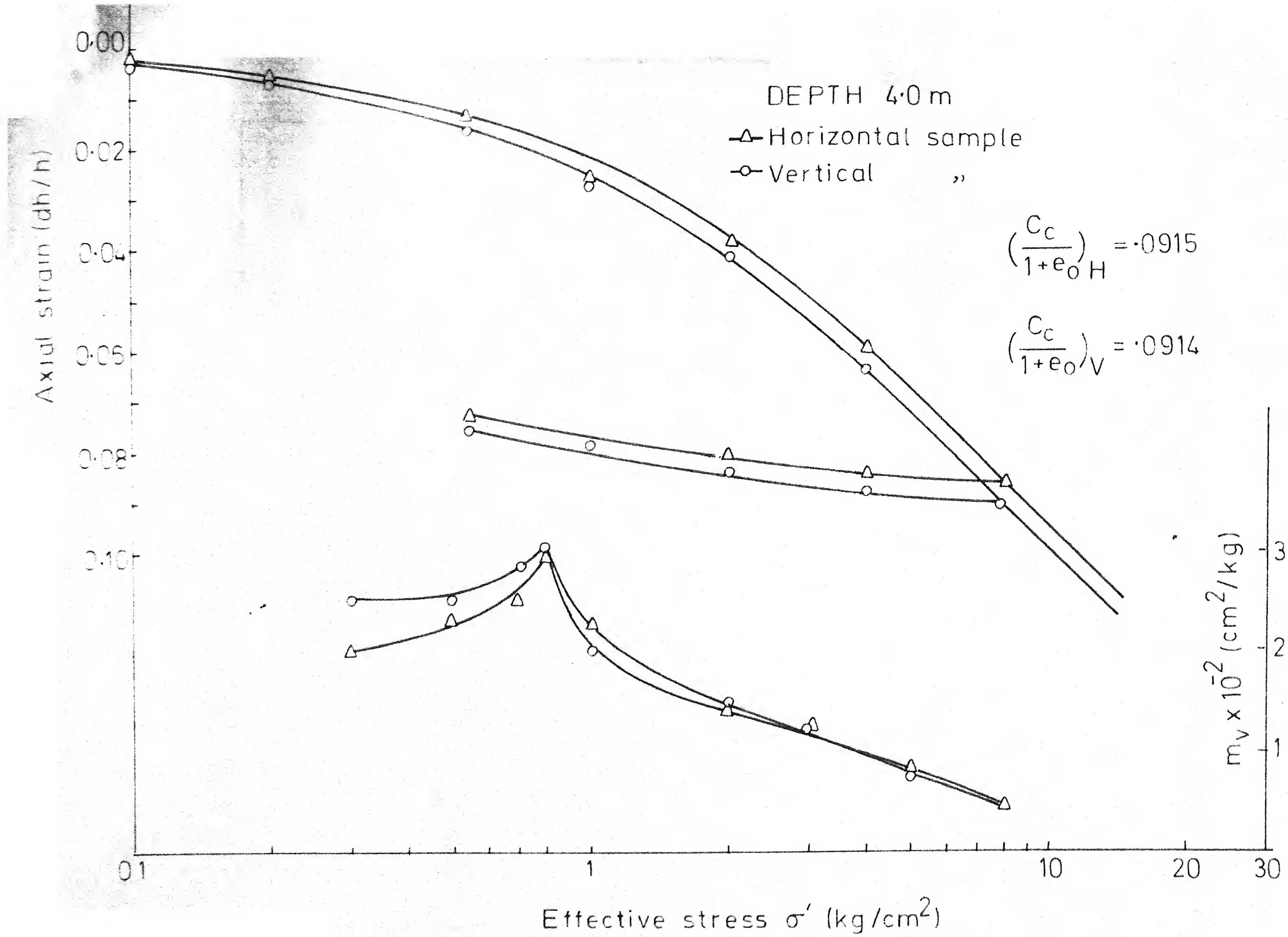


FIG. 3·7 CONSOLIDATION CHARACTERISTICS

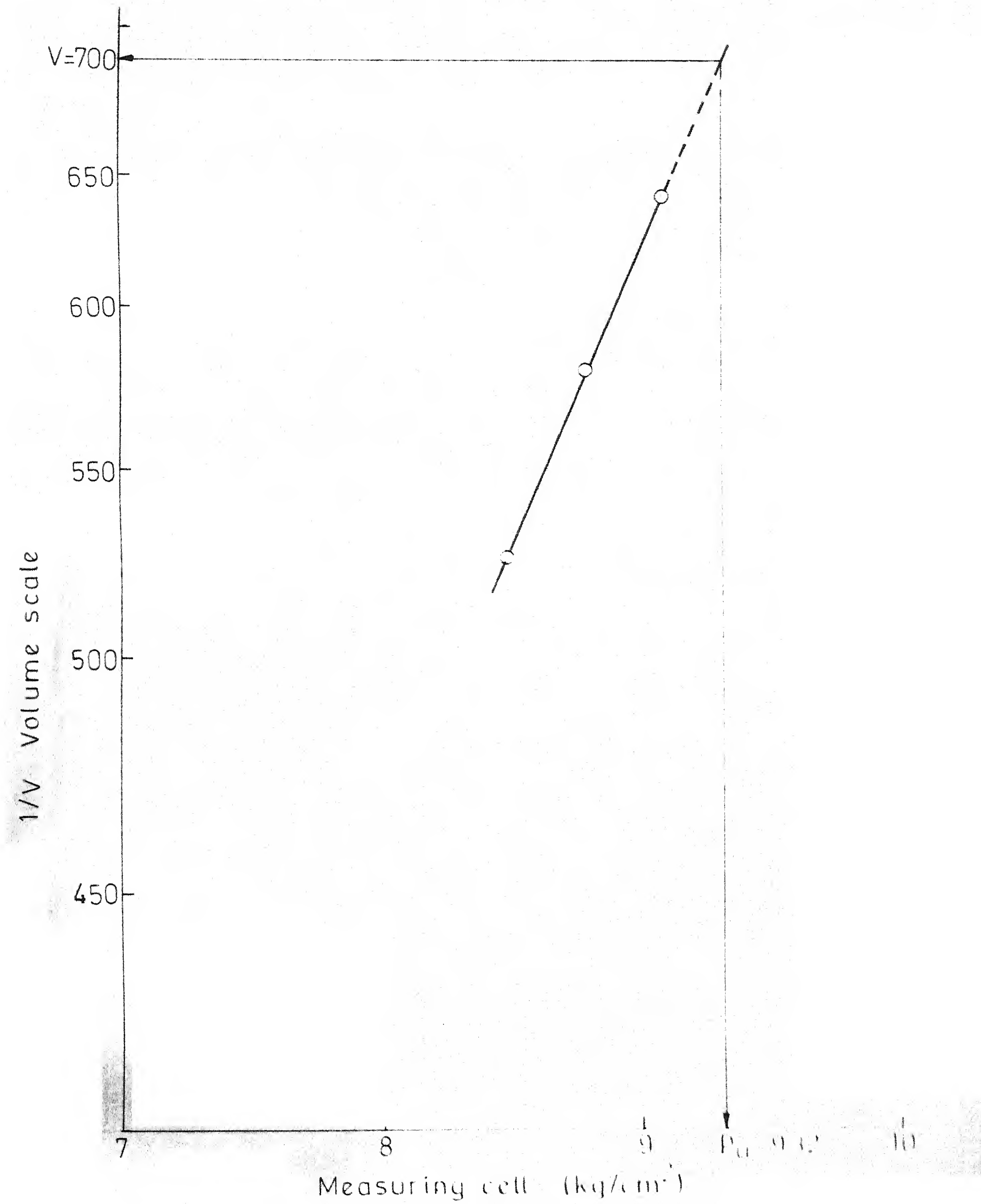


FIG. 3·8 PLASTIC PHASE CURVE ON A P VS 1/V PLOT

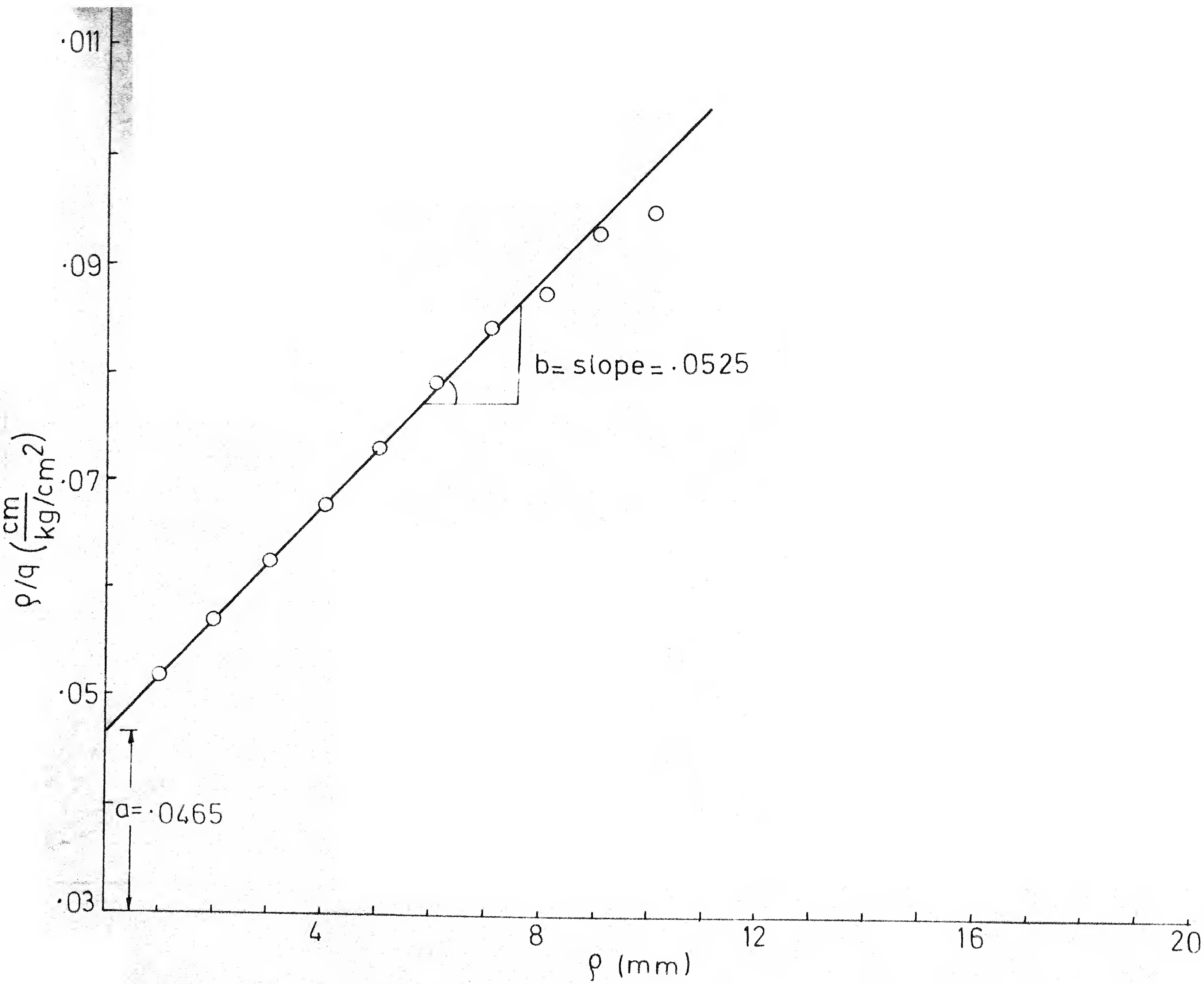


FIG 3·9 SETTLEMENT VS SETTLEMENT / LOAD INTENSITY PLOT

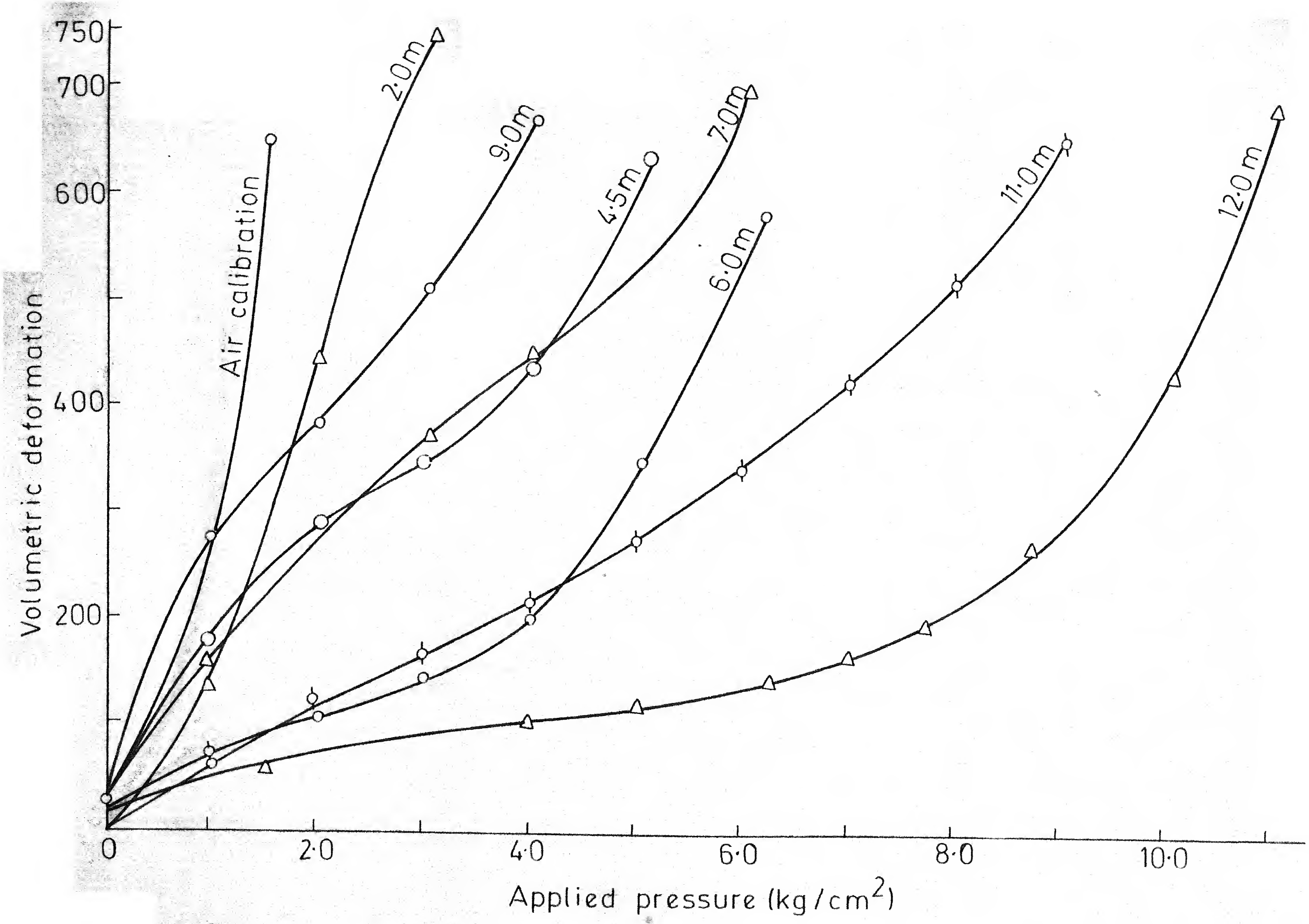


FIG. 4·1 PRESSURE METER CURVE FROM SAME BORE HOLE
(S. Venkatesan, 1980)

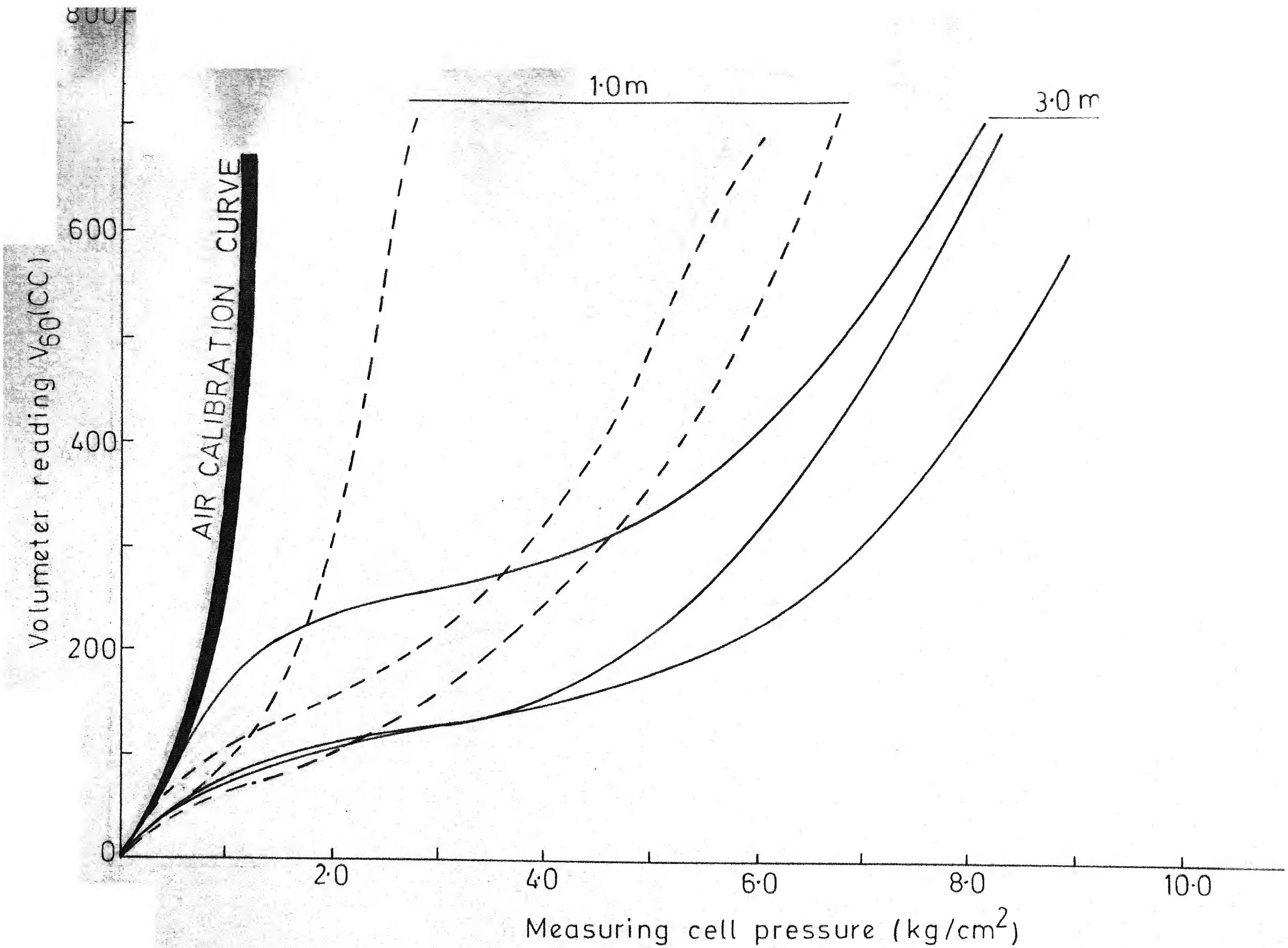


FIG. 4·2 RESULTS OF PRESSUREMETER TEST AT SAME DEPTH

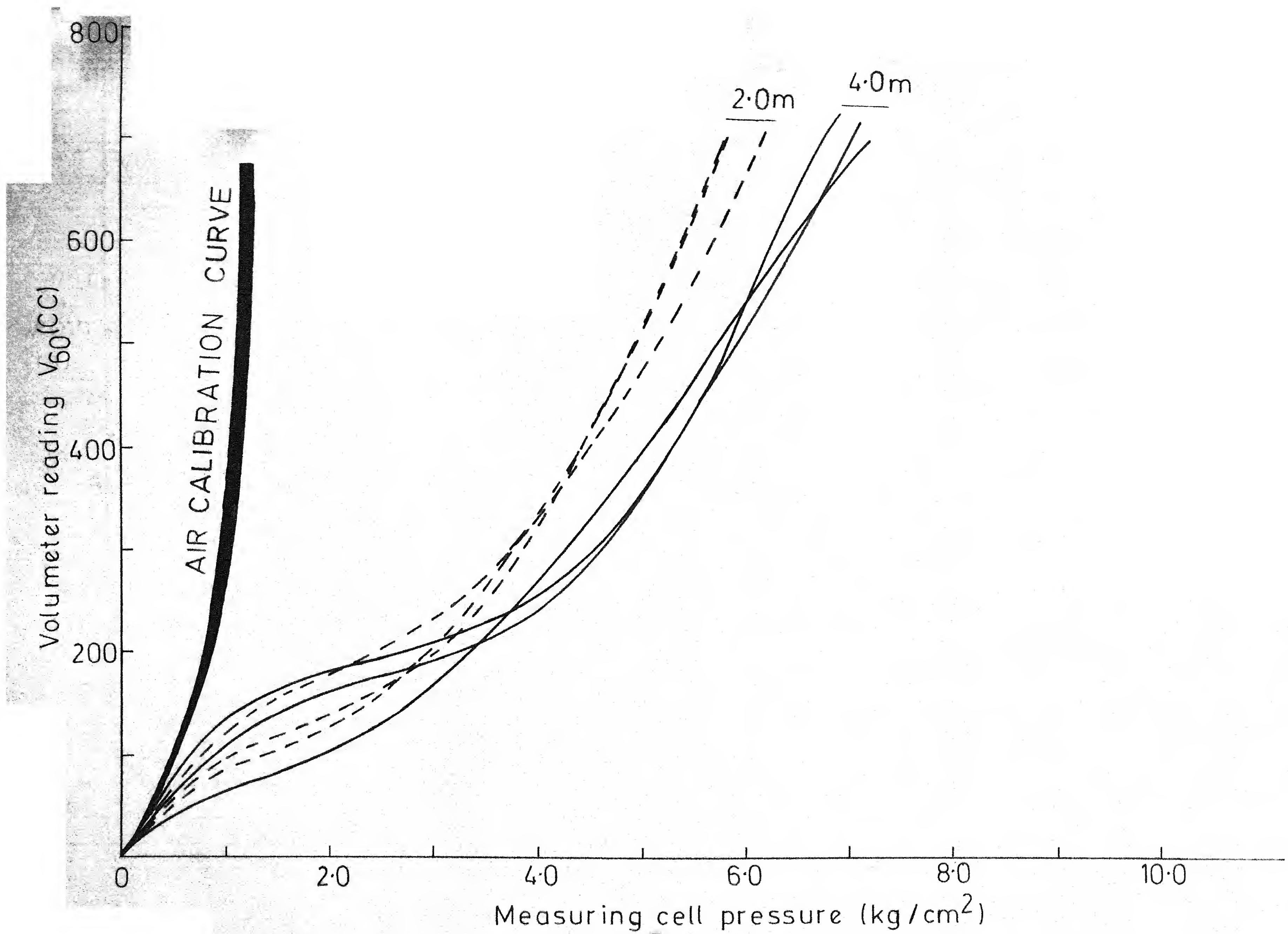


FIG. 4·3 RESULTS OF PRESSUREMETER TEST AT SAME DEPTH

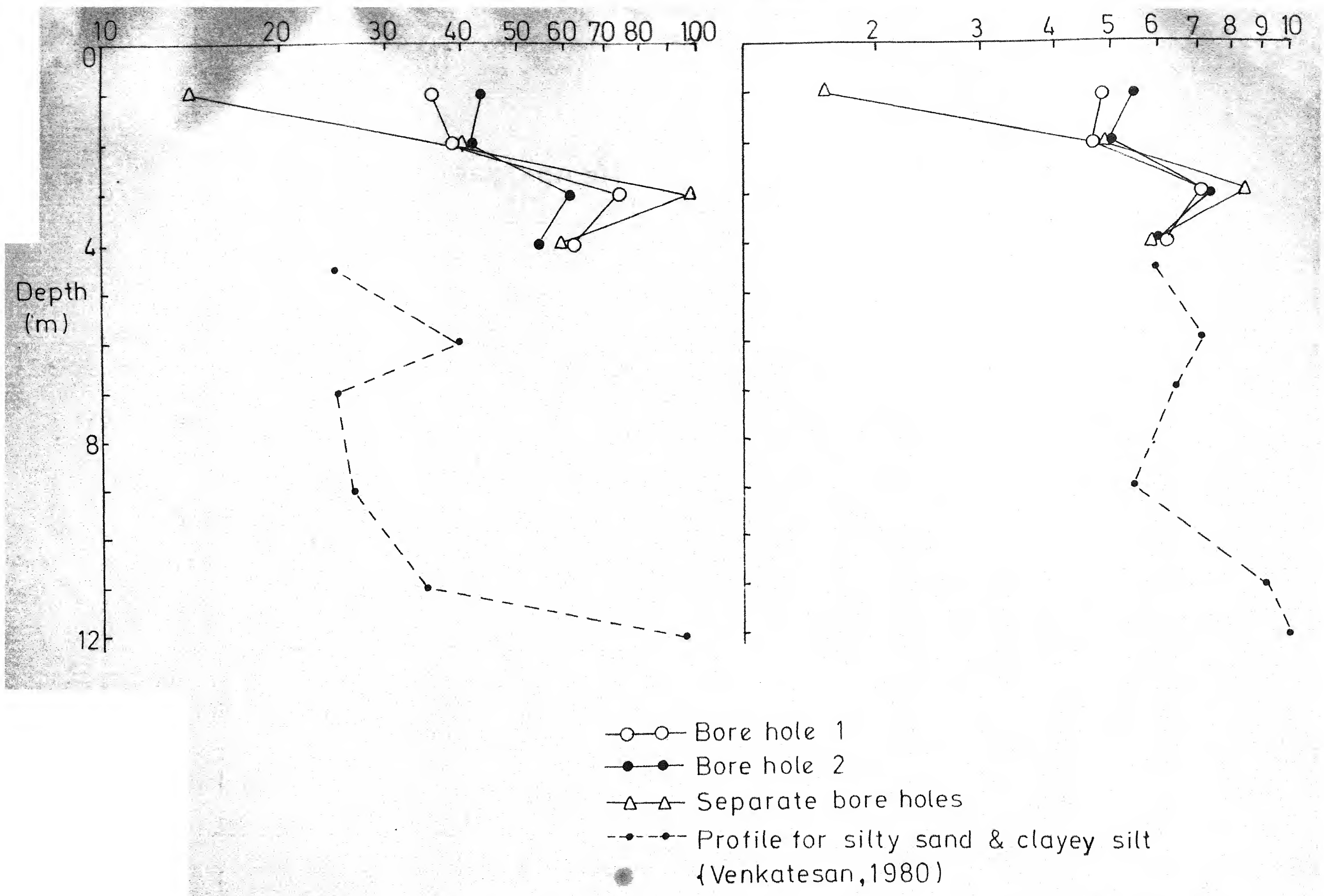


FIG. 4·4 VARIATION OF DEFORMATION MODULUS & LIMIT PRESSURE

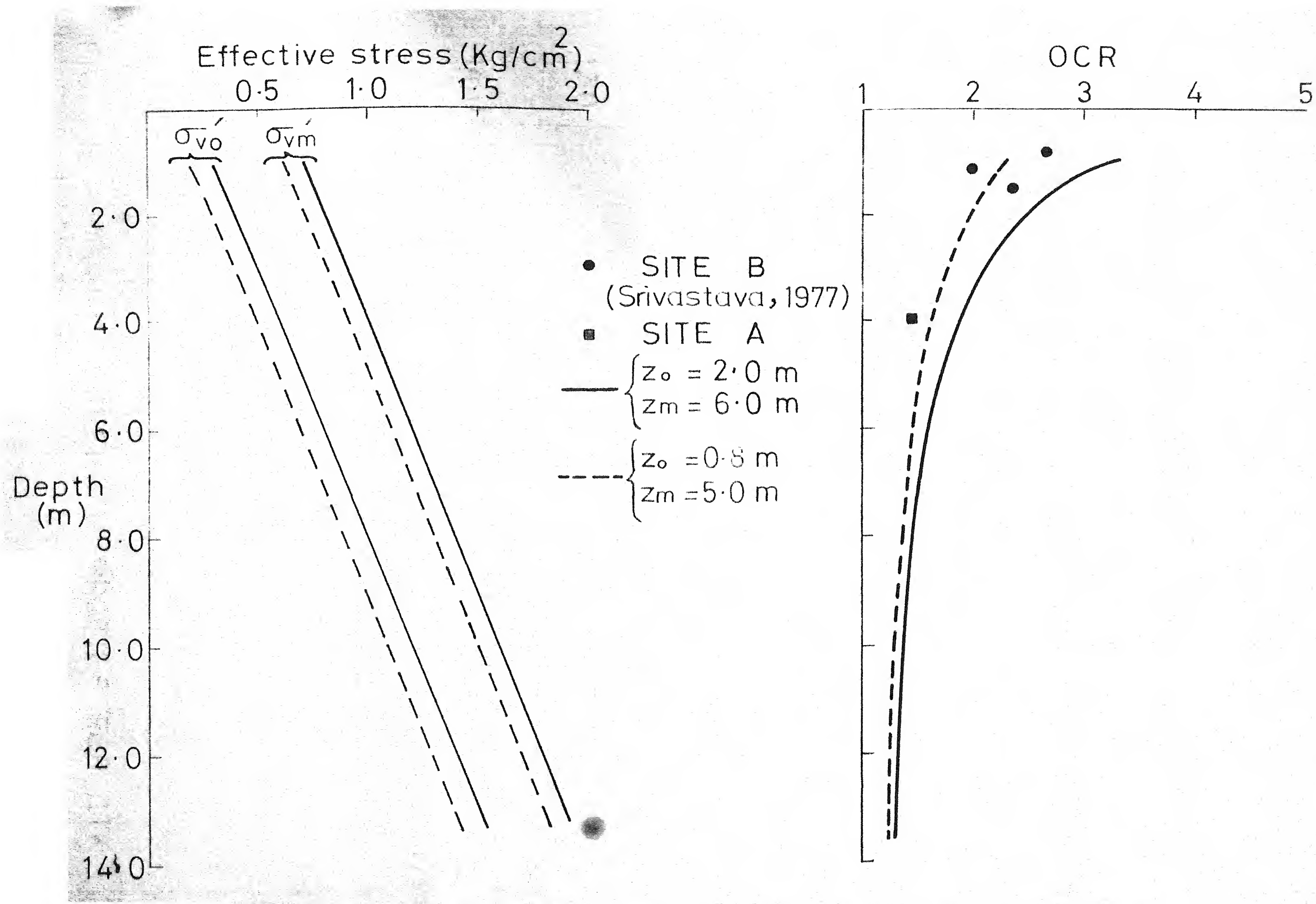


FIG. 4·5 VARIATION OF $\overline{\sigma_{vo}}'$, $\overline{\sigma_{vm}}'$ & OCR WITH DEPTH

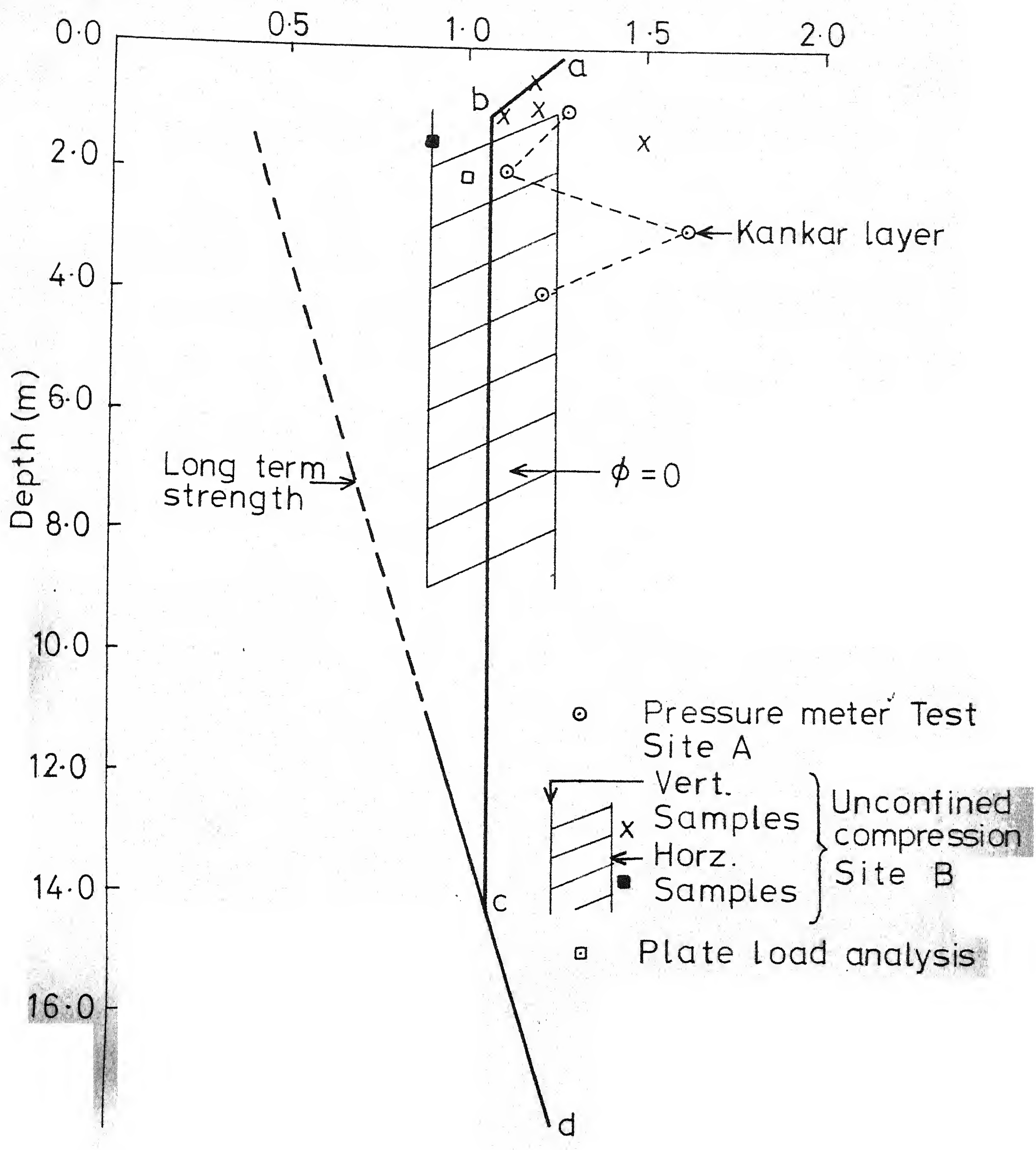


FIG. 4·6 UNDRAINED STRENGTH PROFILE FOR IITK CAMPUS SOIL

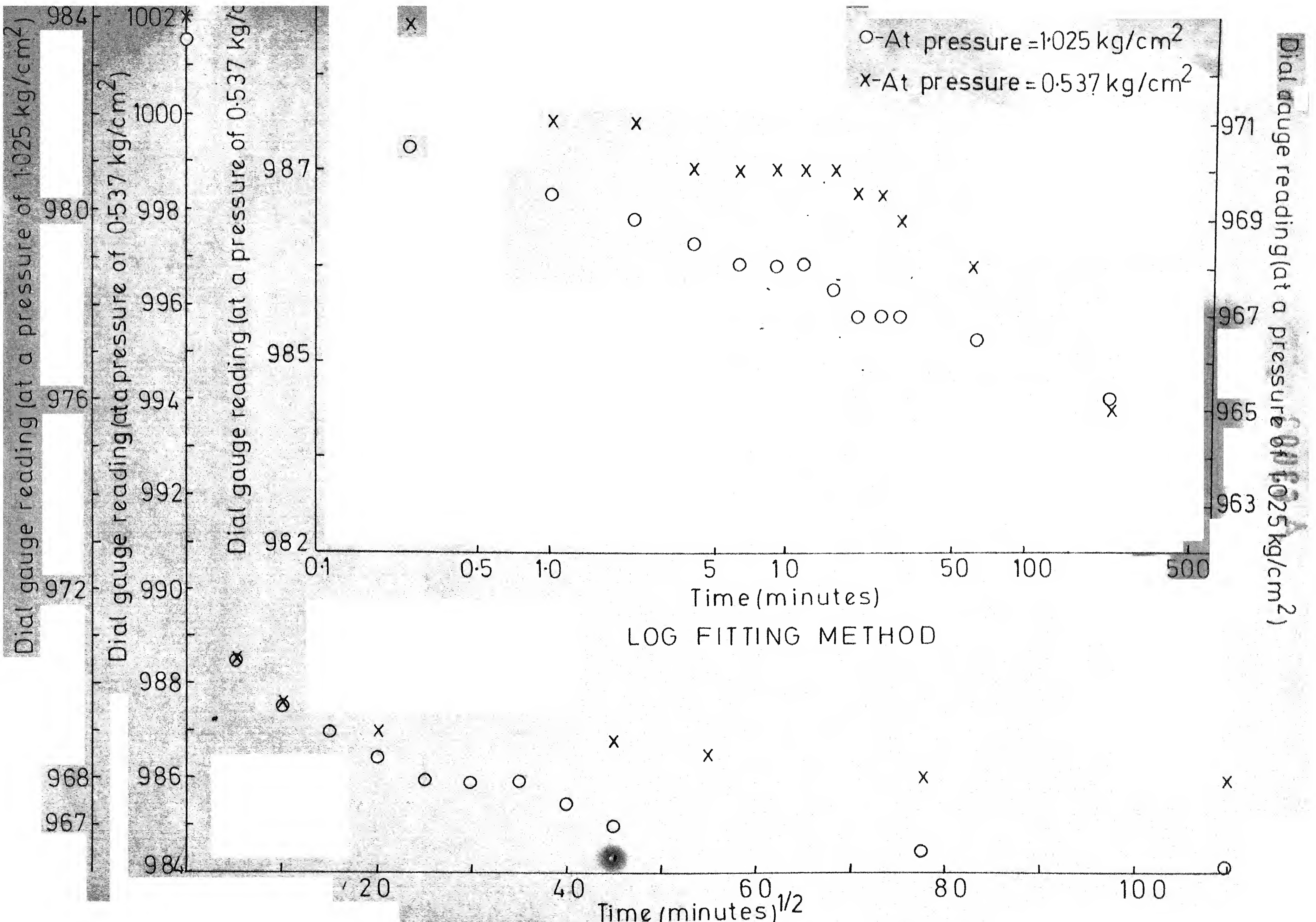


FIG. 4.7 SQUARE ROOT FITTING METHOD